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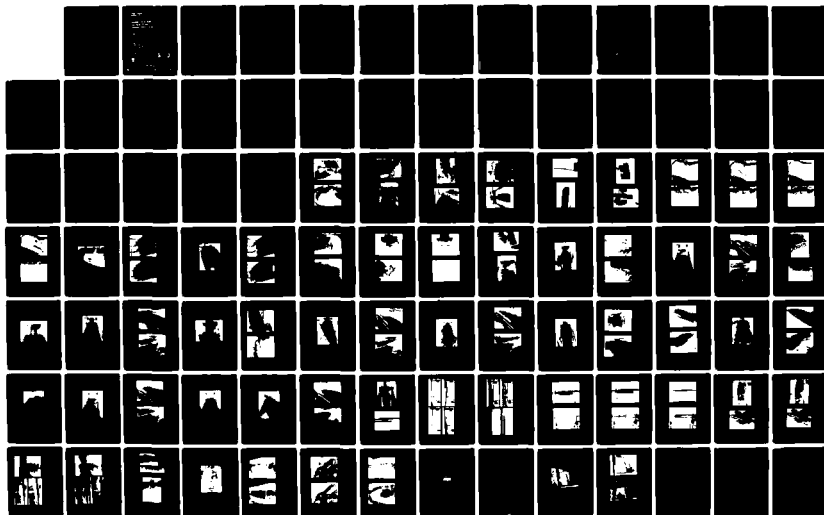
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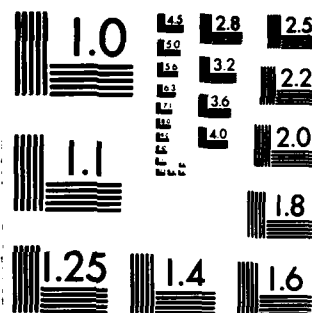
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**HYDRAULIC MODEL  
INVESTIGATION**

**TECHNICAL REPORT NO. 171-1**

**Applegate Dam Spillway  
and Outlet Works  
Applegate River, Oregon**

**SPONSORED BY  
U. S. ARMY CORPS OF ENGINEERS  
PORTLAND DISTRICT**

**CONDUCTED BY  
DIVISION HYDRAULIC LABORATORY  
U. S. ARMY CORPS OF ENGINEERS  
NORTH PACIFIC DIVISION  
BONNEVILLE, OREGON**

**SEPTEMBER 1984**



**U.S. Army Corps  
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) The Applegate Dam project, located in southwestern Oregon, provides flood control storage and includes facilities for upstream fish migration and temperature control of normal project releases. This report presents the results of model studies used in the final design of the project spillway and flood control outlet. Tests with the spillway model indicated that the originally designed approach		

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channel invert could be raised 20 feet and still maintain satisfactory spillway operation. The reduction in approach-channel depth resulted in a considerable decrease in excavation costs. The model was used to evaluate the performance of the spillway plunge pool.

Initial operation of the outlet model revealed the existence of an unacceptable roostertail emanating from the nose of the splitter wall separating the two outlet entrances. The model was used to develop a pier modification which reduced the roostertail to acceptable limits. The report also presents test results used in evaluating performance of the outlet works stilling basin which consists of a unique, dual-chamber hydraulic-jump basin which also functions as an entrance pool for the fish passage facility. ✓

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# PREFACE

Model studies of Applegate Dam spillway and outlet works were authorized by Office, Chief of Engineers, on 20 June 1977 at the request of the U.S. Army Engineer District, Portland (NPP). Studies were conducted at the Division Hydraulic Laboratory, U.S. Army Engineer Division, North Pacific, during the period September 1978 to July 1981.

The studies were conducted by Mr. R. L. Johnson under the supervision of Mr. P. M. Smith, Director of the Laboratory. This report was prepared by Mr. M. M. Kubo, Hydraulics Section, Seattle District.

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## TABLE A

PHOTOGRAPHS 1 TO 59

PLATES 1 TO 48

CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI)  
UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
feet	0.3048	metres
miles	1.609344	kilometres
feet per second	0.3048	metres per second
cubic feet per second	0.02832	cubic metres per second
pounds (mass)	0.4535924	kilograms

APPLEGATE DAM SPILLWAY AND OUTLET WORKS  
APPLEGATE RIVER, OREGON

Hydraulic Model Investigations

PART I: INTRODUCTION

The Project

1. The Applegate Dam is located on the Applegate River 23\* air miles southwest from Medford, Oregon, and 45.7 river miles upstream from the confluence of the Applegate and Rouge Rivers (figure 1). A general plan of the project is shown on plate 1. The project provides 65,000 acre-feet of flood control storage and includes facilities for upstream fish migration and temperature control of normal project releases. The embankment crest is at elevation 2000\*\*. A two-bay spillway having a crest length of 100 feet and elevation of 1943.7 is located to the left of the left abutment. Flood control and normal releases will be accomplished by means of a gated conduit through the embankment dam.

2. The spillway consists of a deep, trapezoidal-shaped approach channel, two-bay, ogee-shaped control structure surmounted by 50.0-foot-wide by 45.6-foot-high tainter gates and a 110-foot-wide rectangular chute terminating in a flip bucket. The chute follows the natural rock line. A secondary flip bucket is located 36 feet below the primary bucket to throw small discharge releases beyond the toe of the structure. A preformed plunge pool is located in the spillway jet trajectory impact area. The spillway is designed to discharge 93,600 cfs at a head of 43.3 feet and a maximum pool elevation of 1987. Model studies showed that the discharge of 93,600 cfs could be passed with a pool elevation of 1985 (actual to design head ratio of 0.95).

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\* A table of factors for converting U.S. customary units of measurement to metric (SI) units is shown on page iii.

\*\* All elevations are in feet above National Geodetic Vertical Datum.

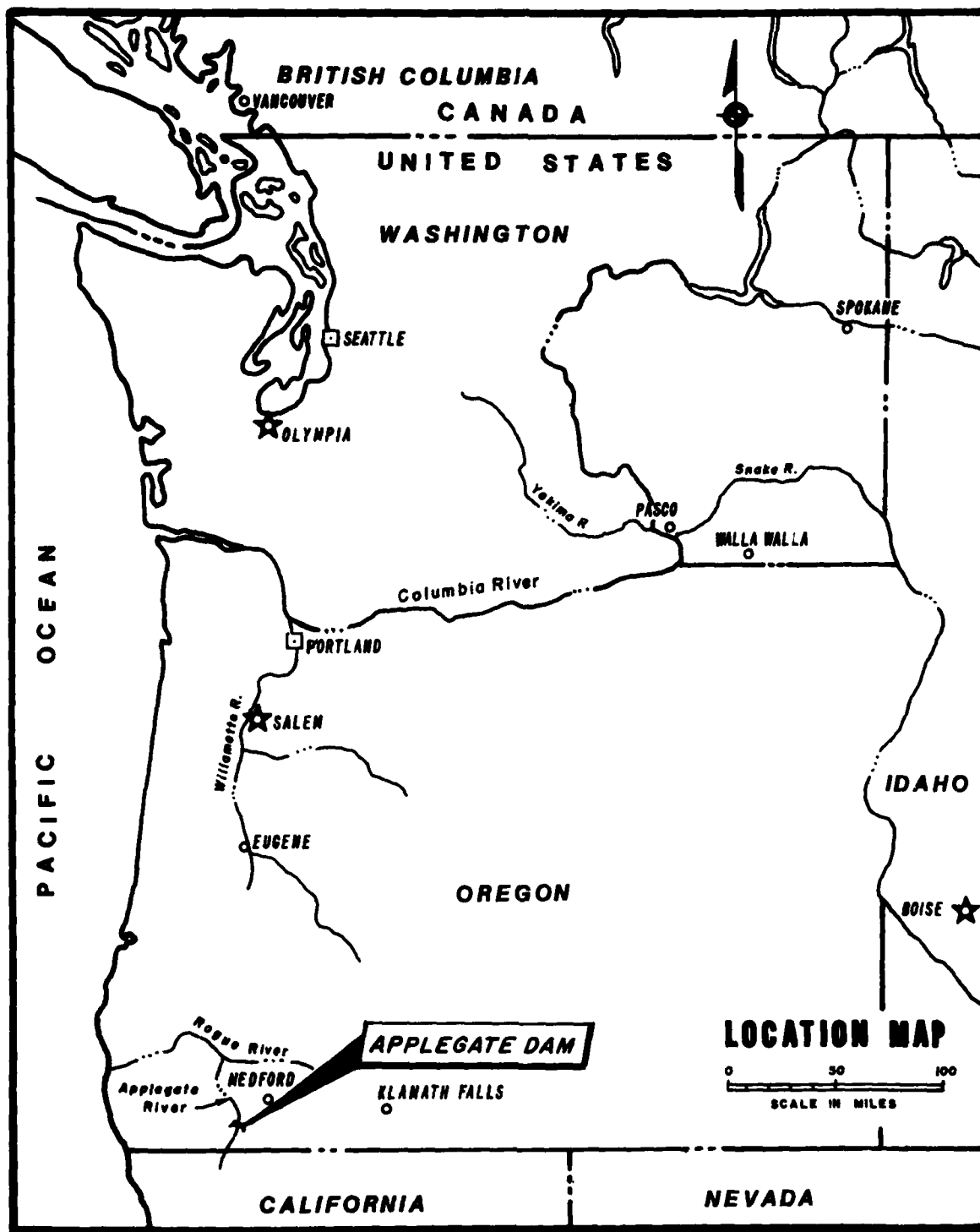


Figure 1

3. The regulating outlet is designed to pass 6,100 cfs at maximum regulated pool elevation 1987. The outlet works consist of a free-standing intake tower, two 5-foot by 7-foot entrance passages controlled by vertical slide gates, an 800-foot-long 9-foot by 14.5-foot oblong conduit, a transition between the dual entrances and conduit, and a dual-chamber hydraulic jump stilling basin designed to also function as an entrance pool for a fish passage facility. The intake tower includes five gated 4-foot by 5.5-foot intake ports which provide temperature control of outflow during normal operation.

#### Purpose of Model Study

4. The spillway design incorporates various features which necessitated model analysis for final design. The effect of the deep, curving approach channel on spillway discharge and flow characteristics was studied in the model and led to development of a shallower, more economical approach channel. The model was used to evaluate the hydraulic jump and sweepout conditions in the spillway flip bucket at lower discharges and study the jet trajectory and impact conditions in the plunge pool at high discharges. The outlet model was used to study general flow conditions and pressures in the outlet conduit and to evaluate the performance of the stilling basin and fish facility entrance conditions. Construction of the project was underway at the time of the model studies. Some studies to refine the hydraulic performance of the structures that might have been made earlier in the design development were not made when the performance of the tested design was acceptable. The cost of design changes would have exceeded the savings of slightly smaller structures or less excavation.

## PART II: THE MODELS

### Description

5. The 1:30-scale spillway model (see plate 2 and photographs 1 through 6) reproduced the left abutment of the dam for a distance of 1,000 feet upstream of the axis, a portion of the adjacent reservoir to include the outlet tower and service bridge, the spillway, the outlet stilling basins, the spillway and outlet exit channels, and 900 feet of the river downstream from the exit channels. The entire spillway was constructed of plastic, while all other structures were constructed of waterproofed wood. Forebay and tailrace topography consisted of hand-molded concrete between sheet metal templates. Crushed rock and stippled concrete were used to reproduce roughness in the forebay and tailrace. Water depth downstream from the model was controlled by an adjustable tailgate.

6. The 1:25-scale outlet model (see plate 3 and photographs 7 through 11) included the conduit, both regulating valves, the stilling basin, the portions of the fish facility, and the exit channel. A large, corrugated metal tank was used as a forebay to reproduce the required head on the outlet. The Applegate outlet entrance design was not reproduced in the model--instead the bellmouth and valves from a previous model (Ririe project) were used. The Ririe design is very close to that used at Applegate and is considered to satisfactorily simulate the Applegate approach conditions, especially with full valve operation. The model structures were constructed of plastic except for the exit channel and portions of the fish facility which were constructed of waterproofed plywood. The smooth plastic pipe used in the model simulated model Reynolds numbers of flow ranging from  $3.9$  to  $6.2 \times 10^5$  and Darcy-Weisbach 'f' values ranging from 0.014 to 0.0128; the same 'f' values expected in the prototype assuming a maximum design effective roughness " $k_s$ " of

0.002 feet with Reynolds number of flow ranging from  $4.9$  to  $7.8 \times 10^7$ . Thus, the full length of the oblong conduit was reproduced to simulate the maximum design value of effective roughness. A minimum effective roughness value of  $0.0002$  was simulated in the model by increasing the head to create the computed depth of flow near the outlet portal. Crushed rock was glued to the exit channel side slopes to simulate riprap.

7. Water used in the operation of both models was supplied by recirculating systems, and discharges were measured by calibrated orifices in the supply lines. Water surface elevations were measured by point gages and water manometers connected to piezometers. Standard laboratory instruments and procedures were used to measure velocities and current directions. Photographs were used to analyze the flow conditions in the spillway approach channel, the plunge pool, and the confluence of the outlet channel with the spillway channel.

#### Scale Relationships

8. The required similitudes of the models to the prototype were obtained with the following scale relationships based on the Froude model law:

<u>Dimension</u>	<u>Ratio</u>	<u>1:30-Scale Relationship</u>	<u>1:25-Scale Relationship</u>
Length	$L_r$	1:30.0	1:25.0
Area	$A_r = L_r^2$	1:900.0	1:625.0
Velocity	$V_r = L_r^{1/2}$	1:5.477	1:5.0
Time	$T_r = L_r^{1/2}$	1:5.477	1:5.0
Discharge	$Q_r = L_r^{5/2}$	1:4929.5	1:3125.0
Roughness	$N_r = L_r^{1/6}$	1:1.763	1:1.71

### PART III: SPILLWAY TESTS

9. Primary spillway discharges observed were 18,400, 29,700, 43,900, and 93,600 cfs. Coupled with the maximum design regulating outlet release of 5,900 cfs, these spillway flows provide river discharges of 24,300 (100-year flood), 49,800 (Standard Project Flood (SPF)), and 99,500 cfs (Spillway Design Flood (SDF)). In addition, spillway discharges ranging from 2,000 to 7,000 cfs were observed for specific areas of interest such as the right side of the approach channel, lower chute, flip bucket, and upstream end of the plunge pool. Pool elevation was measured at the intake structure where prototype measurements would be made and where the velocity head of approach flow was negligible (gage 7, plate 2).

#### Approach Channel

##### Original Design

10. The original-design approach channel (photograph 2) was trapezoidal shaped and had a bottom width of approximately 126 feet with an invert elevation of 1890. It provided a height of spillway crest (53.7 feet) to design head (43.3 feet) ratio of 1.24. A portion of the embankment fill at the left abutment extended upstream forming a re-entrant condition along the right bank of the approach channel. Flow conditions in the approach channel and upstream of the spillway crest were acceptable (see photograph 12) and indicated a potential for raising the invert without compromising acceptable hydraulic performance. The spillway passed the design discharge of 93,600 cfs with a pool elevation of 1985, 2 feet lower than that expected.

### Modifications

11. The approach channel invert was raised to elevations 1900 and 1910 and tested with the spillway design discharge. Hydraulic performance was acceptable under both conditions (see photographs 13 and 14). The tests indicated that the flow conditions would be good also with a higher invert; however, none was tested because part of the channel had already been excavated to elevation 1910. Removal of the embankment fill at the left abutment to a rock bench at elevation 1970 increased local velocities at the fill from 5 fps (occurring with the fill) to 8 to 11 fps (during SDF conditions) and created generally poorer surface flow conditions upstream of the spillway crest (see photograph 15). Removal of the fill had little noticeable effect on flow through the spillway and in the chute.

### Final Design

12. The final-design approach channel was excavated to elevation 1910 to provide a height of crest to design head ratio of 0.78 and included the original-design re-entrant embankment fill at the left abutment (see photograph 16). Plates 4 to 6 show approach channel velocities for SDF discharges of 18,400, 43,900, and 99,500 cfs, respectively. Surface flow conditions with the SDF are shown in photograph 14. Flow around the re-entrant fill on the inside of the channel curve with the SDF had a maximum velocity of 13 fps at the toe of the upstream edge, although general velocities in this area were about 5 fps. An eddy formed around the fill but did not cause adverse effects on spillway flow conditions and was not considered objectionable.

13. Tests were accomplished to evaluate flow conditions around the outlet intake structure bridge piers (plate 7). Plate 8 shows velocities and flow conditions around the bridge piers with the SDF condition. Maximum velocity around the piers was 5 fps, with a maximum velocity of 2 fps near the pier footing.

## Crest and Chute

### Original Design

14. Details of the spillway crest and chute are shown on plate 9. The original-design chute walls were 22 feet high for their entire length. A rooster tail formed downstream from the spillway pier as shown by photograph 17. The rooster tail, which was stable and isolated to the center of the chute, did not cause problems along the chute walls. With ungated spillway operation, the slightly asymmetric flow to the spillway (see photograph 14) caused a low standing wave in the chute from the right bay to the left wall of the chute at station 14+30 and a 3-foot higher water surface profile along the left wall than the right wall.

### Modifications

15. A 32-foot-long pier extension (photograph 18) was developed that greatly reduced the rooster tail (photograph 19); however, the modification did not improve flow conditions in the chute. A vertical wall 35 feet above the approach channel invert extending 50 feet upstream from the right abutment (photograph 20) improved the inflow to the spillway and caused a 2-foot lower water surface profile (SDF) on the left wall of the chute between station 14+35 and station 14+50. Water surface profiles in the chute with the pier extension and both with and without the forebay wall are shown on plates 10 and 11, respectively. The changes in spillway operating performance resulting from both modifications were considered to be largely aesthetic, and neither modification offered a significant enough improvement to warrant their additional cost. Passage of the SDF through the spillway with a pool elevation 2 feet lower than maximum design pool indicated that the spillway could have been slightly smaller. Because a portion of the spillway area had already been excavated, tests to refine the spillway size were not made.

### Final Design

16. The final-design spillway and chute profile is shown in the center of plate 9. The chute walls were generally lowered through their entire length based on the model water surface profiles. The water surface profiles on the final-design chute wall with the SDF are shown on plate 12. The spillway passed the 93,600-cfs spill of the SDF at pool elevation 1985--2 feet lower than the designed maximum pool elevation 1987. With pool elevation 1987, the spillway free-flow discharge was 102,300 cfs.

17. Spillway operation did not create surging conditions in the relatively shallow approach channel. Tests to evaluate potential for surging were accomplished with gated operation at maximum pool elevation 1987, with free flow at pool elevation 1985 (SDF discharge), and with free flow at maximum pool elevation 1987.

18. During prototype excavation of the spillway approach, a zone of fractured rock was uncovered near the right bank just upstream of the spillway crest that required stabilization with a concrete blanket. This modification (see photograph 21) was tested in the model and caused no change in flow conditions. Maximum velocities adjacent to the concrete were 5 to 7 fps.

19. With small discharges a hydraulic jump formed in the downstream mild-slope section of the spillway chute, and flow spilled over the lip of the upper flip bucket and flipped from the lower bucket (photograph 22). With maximum pool elevation 1987, the upper limit of rising gated discharges at which the jump would sweep out and flow would flip from the upper bucket was 7,100 cfs. The lower limit of receding gated discharge at which flipping at the upper bucket would occur was 4,300 cfs. With free flow the upper and lower limits were 8,300 and 6,900 cfs, respectively. Photographs 22, 23, and 24 illustrate flow conditions at which a discharge of

7,000 cfs flipped from the lower and upper buckets and 4,500 cfs flipped from the upper bucket, respectively. With a discharge of 7,000 cfs, a tailwater elevation of 1772.4 (at gage 3, plate 2) was required to drown the flip action from the lower bucket. This tailwater elevation is 6 feet higher than the theoretical elevation for a river discharge of 12,900 cfs, indicating that the lower bucket invert elevation was satisfactory.

#### Plunge Pool and Downstream Channel

##### Original Design

20. The original-design plunge pool and downstream channel are shown on plate 13 and photograph 25. The plunge pool was somewhat arbitrarily sized with the intention that it should prevent significant erosion from occurring with spillway discharges up to 29,700 cfs (200-year-frequency flood). No attempts were made at movable-bed modeling in the plunge pool area--instead, standard hydraulic characteristics--velocities, wave heights, and impact pressures--were determined in the fixed-bed model from which judgmental performance of the plunge pool could be made.

21. Flow conditions in the plunge pool and downstream channel with spillway discharges of 4,500 to 93,600 cfs are shown on plates 14 to 20 and photographs 26 to 29. With discharges up to the 200-year-frequency flood, maximum velocity in the plunge pool was 23 fps and occurred along the left side of the plunge pool where the excavated rock was to be protected with a concrete cover. Velocities as high as 28 fps occurred at the top of the transition slope between the plunge pool and downstream channel. Wave rideup at the sides of the plunge pool adjacent to the impact area was 10 to 11 feet. Impact pressures on the bottom of the plunge pool were as high as 65 feet of water. The flip trajectory (see plate 21) impacted near the downstream end of the plunge pool.

22. With the SPF discharge, the jet trajectory impacted on the upstream edge of the downstream channel and very little flow action occurred in the plunge pool. Velocities along the sides of the channel downstream from the impact zone were 17 to 25 fps.

23. The SDF trajectory impacted on the upper part of the transition slope, the toe of the adjoining right side slope, and on the upstream end of the channel. An eddy formed along each side of the plunge pool with maximum velocities of 12 fps. The maximum wave rideup was 20 feet on the left side of the pool just upstream from the impact area. Maximum velocity along the sides of the downstream channel was 63 fps. A standing wave formed near the downstream end of the channel. The sweep-out effect of the flow caused the depth in the outlet channel (gage 5) to be lower than with the 200-year flood and the SPF.

24. With the larger discharges (100-year frequency and above) low-velocity flow flooded the fish facility, although water did not reach the embankment toe with any of the discharges tested.

#### Modifications Tested

25. In an effort to reduce the highly turbulent flow and high velocities at the transition from the plunge pool to the downstream spillway channel, the bank transition was lengthened from 50 to 100 feet (plate 22 and photograph 30). Short (16.4 feet long) walls (see plate 22 and photograph 30) were added along the sides of the plunge pool at the toe of the flip buckets to provide protection against erosion in the event that debris material might collect in the upstream corners of the plunge pool.

26. Flow conditions with the revised transition for spillway discharges of 29,700 to 93,600 cfs are shown on plates 23 to 25. The longer transition did not noticeably improve flow conditions.

Local velocities changed, but maximum velocities were essentially the same as those which occurred with the shorter transition. Wave action was greater with the modified transition. The addition of the walls at the upstream end of the plunge pool did not affect flow conditions in the plunge pool.

27. An 80-foot longer plunge pool (Plan B and photograph 31) was evaluated with spillway discharges of 29,700 (200-year frequency) and 43,900 cfs (SPF). Flow conditions for these tests are shown on plates 26 and 27 and photographs 32 and 33, respectively. Flow conditions in the longer plunge pool suggested a marked improvement in stilling action would occur with this design; however, NPP concluded that the additional excavation was not economically justified.

#### Final Design

28. The final-design plunge pool included the short concrete walls at the toe of the flip bucket and the 50-foot-long transition between the plunge pool and downstream channel. The original-design plunge pool length was retained.

29. Passage of the 200-year flood spill (29,700 cfs) through only the right bay of the spillway was observed with both the final-design plunge pool and downstream channel and with the Plan B plunge pool. The flow from the right bay spread across the chute in the steep section, and most of it flipped from the left side of the upper bucket. The flow conditions in the plunge pool and the upstream end of the channel are shown for both designs on plates 28 and 29 and in photographs 34 to 37, respectively. In the final-design plunge pool, the trajectory impacted on the bottom transition slope with the greatest impact and highest velocities occurring in the left side of the pool. The maximum velocity was 30 fps on the transition slope at the left edge of the impact area. Maximum wave rideup on the sides of the plunge pool was 12 feet at both sides of

the impact area. In the Plan B plunge pool, the trajectory impacted on the bottom of the pool 35 feet from the bottom transition. As with the shorter pool, the maximum velocity was 30 fps and occurred at the upstream end of the channel as well as at the left side of the impact area. Maximum wave rideup was 3 feet greater than with the final design.

## PART IV: REGULATING OUTLET TESTS

### Conduit

#### Original Design

30. Details of the original-design outlet are shown on plate 3. The splitter wall between the two converging conduit passages downstream of the valves (station 10+00.654 to station 10+86.65, plate 2) contains a low-discharge pipe exiting into the main conduit. The pipe, though not reproduced in the model, necessitated that the splitter wall be 6 feet wide at the downstream end (see photograph 38). The converging flow from the two passages formed a rooster tail at the end of the wall that extended into the upstream end of the oblong conduit section. When the maximum discharge of 6,100 cfs was passed (with fully open valves and maximum pool elevation 1987), the rooster tail was large and caused the dissipation of considerable energy within the tunnel (photograph 39). The tunnel flowed almost full as crests of waves passed through the downstream end (photograph 40), and the depth of the outflow fluctuated 7.3 feet. Discharge into the primary stilling basin was variable, and waves overtopped the basin walls.

#### Final Design

31. Ten modifications to the end of the splitter wall were studied in the development of a design to damp the rooster tail and improve tunnel outflow. The modifications were of two general types--tapered extensions and deflectors on the walls (plate 30). The extensions damped the rooster tail but not enough to improve the tunnel outflow adequately. Plans 3 and 4--deflectors that changed the path of the full depth of the flow slightly--caused no improvement of the outflow. Plans 8, 9, and 10--deflectors that changed

the path of the lower portion of the flow--caused the lower flow elements to converge after the top elements and form almost indiscernible rooster tail. Having the broadest base (1.25 feet), Plan 8 was the most effective in damping the rooster tail and reducing the depth fluctuation at the outlet (photograph 41). The fluctuation was reduced to 1.5 feet. A rooster tail formed at the end of the splitter wall with smaller discharges, but the waves damped in the tunnel and fluctuation of the outflow was minimal. Conditions with discharges of 4,500 cfs (maximum flow with fully open valves and minimum pool elevation 1889), a gated flow of 4,500 cfs with maximum pool, and an ungated flow of 3,050 cfs through only the left valve with maximum pool are shown in photographs 42, 43, and 44, respectively. Flow conditions were also satisfactory with 50- and 75-percent valve openings with minimum and maximum pool levels and with fully open valves during raising and lowering of the pool between minimum and maximum levels. Plan 8 was adopted and used in all other tests of the outlet.

#### Stilling Basin and Downstream Channel

32. The stilling basin (see plate 31) consists of primary and secondary basins with a fish barrier sill separating the basins. The stilling basin is designed for 6,100 cfs with a conduit effective roughness of 0.0002 feet; however, the model conduit effective roughness was 0.002 feet. The stilling basin design condition was simulated in the model by increasing head on the conduit to create the computed depth of 8.98 feet at station 19+28 near the conduit exit.

33. With the conduit design condition of " $k_s$ " equal to 0.002 feet, the stilling basins contained the design discharge of 6,100 cfs (photograph 45). However, with the basin design condition (conduit " $k_s$ " equal to 0.0002 feet), the basin walls downstream from the barrier sill were overtopped (photograph 46 and plate 32).

Occasionally a wave overtopped the walls at the downstream ends of both basins as shown by the maximum water surface profile on plate 32. The wall transition sections between the basins were extended as shown on plate 32 to contain all the flow except for the occasional very high waves. Photographs 47 through 49 show flow conditions in the stilling basins with conduit " $k_s$ " equal to 0.002 feet with the following conditions: 4,500 cfs with maximum pool, 4,500 cfs with minimum pool, and 3,050 cfs with single-valve operation at maximum pool. Pressures on the baffles in the primary basin were positive with all discharge conditions of conduit " $k_s$ " equal to 0.002 feet (table A). With the simulated condition of " $k_s$ " equal to 0.0002 feet, pressures on the baffles were as low as -12 feet of water--an indication of cavitation potential. Since the very smooth finish existing in the model conduit would not be obtained in the prototype, a change in baffle or basin design to eliminate the indicated cavitation potential was considered unnecessary. Average pressures observed in the basin fishway openings W-1 and E-1 are also listed in table A.

34. Flow conditions in the downstream channel were satisfactory with the design discharge of 6,100 cfs when the roughness " $k_s$ " of the conduit was 0.002 feet and the spillway was not in operation (tailwater for a river discharge of 6,100 cfs). The maximum bottom velocity at the end sill of the secondary stilling basin was 20 fps (plate 33). Flow along the right bank was downstream with velocities of 1 to 8 fps; flow along the left bank was upstream with velocities of 2 to 5 fps. Waves were 4 feet high at the upstream end of the channel and rode up to but did not overtop the right bank. When the conduit roughness " $k_s$ " was 0.0002 feet, waves from the stilling basins were increased and occasionally rode up the right bank onto the roadway at the upstream end of the channel. When the river discharge was 10,000 cfs (3,900 cfs spill), the increased tailwater (elevation 1764.9) caused waves to overtop the walls of the secondary basin and the right bank. A cap on the basin walls

with a 1-foot overhang and a curb to elevation 1772 at the top of the right bank contained the wave action. Tailwater with a river discharge of 15,000 cfs was at the top of the right bank, and the roadway and area around the fish facilities were flooded by wave overflow. With the higher tailwater from river discharges of 15,000 and 20,000 cfs, wave action from the stilling basins was damped but not eliminated.

35. The channel downstream from the stilling basin (see photograph 7) had an 80-foot bottom width. Flow conditions downstream from the secondary stilling basin with discharges of 180, 500, 1,000, 2,000, 4,500, and 6,100 cfs are shown on plates 34 to 39. Although eddies and upstream flow occurred along the left bank at all discharges, there was no tendency for sand or gravel to move into the basin. Material artificially placed in the basin was swept out as soon as velocities were high enough to move it. Flow conditions along the right bank were good for fish attraction with a path of downstream velocities ranging from 1 to 8 fps for the discharges tested. Photograph 50 shows the flow condition with a discharge of 6,100 cfs. In an attempt to eliminate the large eddy or slack water area along the left bank, the bank was filled to narrow the channel to a width of 60 feet (photograph 51). Slack water was eliminated, and the size of the eddy was reduced. A small eddy still occurred at the downstream end of the left training wall (plates 40 to 44), and although material was not carried into the basin from this eddy, good fish attraction conditions along the right bank were lost (photograph 52). Upstream or fluctuating flow occurred along the bank for all but the minimum flow (fishway attraction flow only). Although good downstream flow occurred farther out in the channel directly downstream from the fishway entrance this plan was discarded because flow along the channel bank was not adequate for fish attraction.

36. A low sill (see photograph 51) was installed in the downstream channel to create additional depth at the fish collection facilities adjacent to the stilling basins during low river discharges. A 4-foot-high sill across the channel at station 25+00 (247.5 feet from the stilling basins) raised the tailwater at the basins 1.5 feet with a discharge of 6,100 cfs. The additional tailwater caused the hydraulic jump in the secondary stilling to occur farther upstream. Although wave action was damped slightly, the increased depth permitted the waves to ride up onto the roadway and fishway parking area on the right bank. The same sill located at station 23+50 (97.5 feet from the basin) was less effective and the waves were higher than with the downstream location. The sill had an 8-foot-long by 1.5-foot-deep notch at the center to pass very low flows. With a river discharge of 100 cfs passed through the fish facilities and spilled through the downstream entrance (E-2) and the accompanying diffuser, fish attraction flow was good from the notch along the right side of the channel and into the entrance channel. The outflow in the fish entrance channel divided allowing a portion to continue down the channel and a portion to pass through the notch in the channel wall and out through the right side of the secondary basin.

37. The low wall (elevation 1770) between the secondary basin and fish collection channel (see photograph 53) was thought to contribute to wave action in the downstream channel. In an attempt to reduce the wave action, the low wall was raised to elevation 1770 (see photograph 54). Flow conditions in the downstream channel with the raised wall and 6,100-cfs outlet discharge (see plate 45 and photograph 55) were detrimental to fish attraction along the right bank of the channel. The desirable fish attraction velocities which had occurred with the original wall design (1 to 8 fps downstream, see plate 39) changed to upstream flows with a series of eddies separating the flow near the channel bank from the main flow farther out in the channel (see photograph 55 as compared to photograph 50).

The 60-foot-wide channel experienced similar undesirable fish attraction conditions with the higher wall (see photograph 56). The tests revealed that flow from the secondary basin was required to expand over the low wall in order to maintain desirable fish attraction flow along the right bank of the downstream channel; therefore, the originally designed lower wall was retained.

## PART V: FISH FACILITY TESTS

38. The fish collection facility (plate 46 and photographs 10 and 11) is located adjacent to the right wall of the stilling basin. The principal water supply for the facility was drawn from the primary stilling basin through opening W-1 (see plate 46) which had a sill elevation of 1759--5 feet above the floor of the primary stilling basin. Tests were made to determine the maximum outlet discharge at which the withdrawal could be made without excessive waves in the surge pool and fluctuating outflow through openings W-3 and W-4 (plate 46) to the fishway supply conduits. The tests showed that good water supply conditions occurred with all outlet discharges. With the maximum outlet discharge of 6,100 cfs and a withdrawal of 200 cfs, surging in the pool was 0.7-foot high on the wall containing opening W-1 and 0.8-foot high on the opposite wall where the water level at the trashracks was an average of 1.7 feet higher than at the upstream end. The surging was damped in the surge pool, and outflow through openings W-3 and W-4 was constant (photograph 57). With an outlet discharge of 4,500 cfs, (gated with maximum pool or ungated with minimum pool), the maximum fluctuation in the surge pool was 0.5 foot and the water level on the wall opposite opening W-1 was 1.2 feet higher at the trashrack than at the upstream end (photograph 57). With the maximum single-valve outlet discharge of 3,050 cfs, the maximum surge was 0.1 foot and the water level on the wall opposite W-1 was 0.9 foot higher at the trashrack than at the upstream end (photograph 59). Flow was also observed with the sill lowered to the level of the basin floor. Boiling and surging in the surge pool were increased with the lower sill.

39. The District anticipated that entrance E-1 to the fish ladder would have to be closed at high outlet discharges in order to prevent a reversal of flow through the entrance. The effect of closure of E-1 was tested in the model with outlet discharges varying from 500 to 6,100 cfs. Flow conditions with entrance E-1 open

and closed are shown on plates 47 and 48, respectively. Tests indicated that flow would always be out of entrance E-1 with velocities ranging from 7 to 14 fps. Flow surged in and out of entrance E-2 when E-1 was open but flowed out (downstream) at velocities of 6 to 7 fps when E-1 was closed. With E-1 open or closed, flow velocity over the downstream sill of the facility was acceptable with velocities of 3 to 9 fps.

## PART VI: SUMMARY

40. The model was used to verify design of the long, sharply curving spillway approach channel. Test results indicated that the originally designed channel invert could be raised 20 feet--resulting in a considerable decrease in excavation costs and still maintain satisfactory spillway operation. The spillway design discharge of 93,600 cfs passed over the spillway with a maximum pool elevation of 1985--2 feet lower than maximum regulated pool elevation 1987.

41. The model revealed a large rooster tail as a result of expending flow from the center pier of the spillway. Although a pier extension was developed which eliminated the rooster tail, the rooster tail was stable and confined to the center of the spillway chute and did not create adverse flow conditions in the chute. The pier extension was not included in the final design. Model results indicated that a lowering of the originally designed chute walls was acceptable.

42. The spillway plunge pool was studied in a fixed-bed model, and standard hydraulic characteristics of flow conditions within the plunge pool were used in evaluating the performance of the plunge pool. This evaluation concluded that the plunge pool would function satisfactorily (preventing significant erosion) for discharges up to the 200-year flood; however, floods approaching and in excess of the SPF would cause excessive erosion in vicinity of the plunge pool and downstream channel. A plunge pool 80 feet longer than originally designed markedly improved flow conditions and stilling action but was not included in the final design as it was concluded to be economically unjustified.

43. A modification to the regulating outlet splitter wall downstream from the valves was developed through the model to eliminate an unacceptable rooster tail emanating from the splitter wall. Flow

conditions both in and downstream from the outlet stilling basin were studied to evaluate fish attraction characteristics at the fish collection facility. Tests were accomplished with various operating conditions at the fish facility entrances to evaluate effects of such operations.

TABLE A

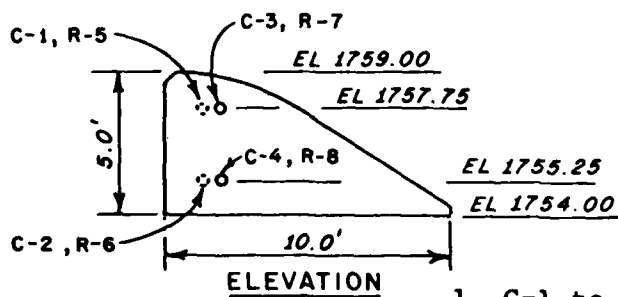
## PRESSURES ON STILLING BASIN BAFFLES AND IN FISHWAY ENTRANCES

Piezometer Number	Regulating Outlet Discharge in CFS				
	3050*	4500**	4500***	6100***	6100 <sup>†</sup>
	Pool Elevation				
	1987	1889	1987	1987	1987
	Pressure in Feet of Water				
C-1	17.3	15.8	12.8	8.4	-0.6
C-2	20.3	19.2	15.8	12.1	-0.7
C-3	17.4	15.7	12.5	8.3	-2.6
C-4	20.3	19.0	15.8	11.8	-3.0
R-5	18.2	17.4	13.5	7.5	-10.4
R-6	20.9	20.4	16.8	11.4	-6.2
R-7	18.4	17.6	13.3	7.0	-12.0
R-8	21.0	20.6	16.6	10.6	-10.3
W-1	14.8	17.0	16.4	16.7	
E-1	2.6	3.0	3.0	3.3	

\* Left valve fully open, right valve closed

\*\* Both valves fully open

\*\*\* Both valves partially open

<sup>†</sup> Both valves fully open, outlet roughness  $k_s = 0.0002$ 

## NOTES

1. C-1 to C-4 in center baffle.
2. R-5 to R-8 in right baffle.
3. Baffle locations shown on plate 31.
4. Locations of entrances W-1 and E-1 shown on plate 46.

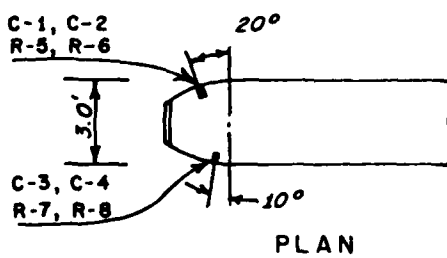


TABLE A

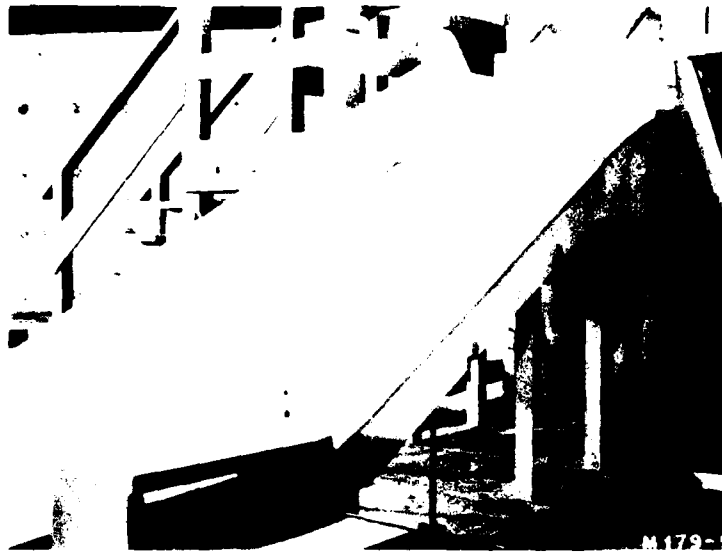


Photograph 1.

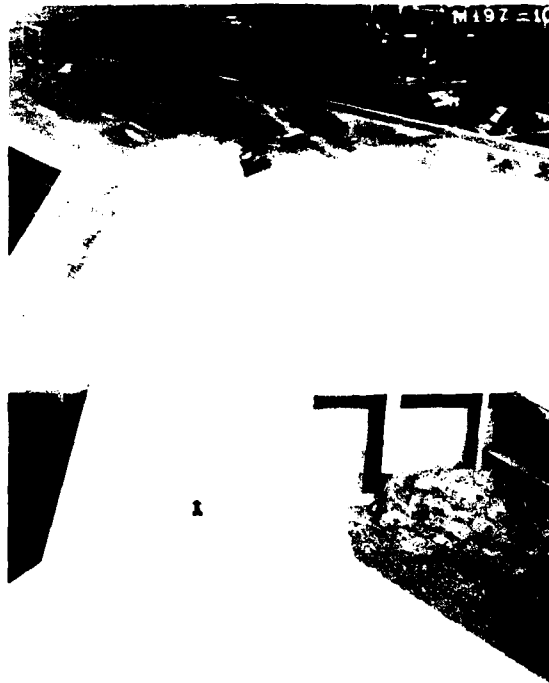


Photograph 2.

Original design forebay and spillway approach channel.



Photograph 3.



Photograph 4.

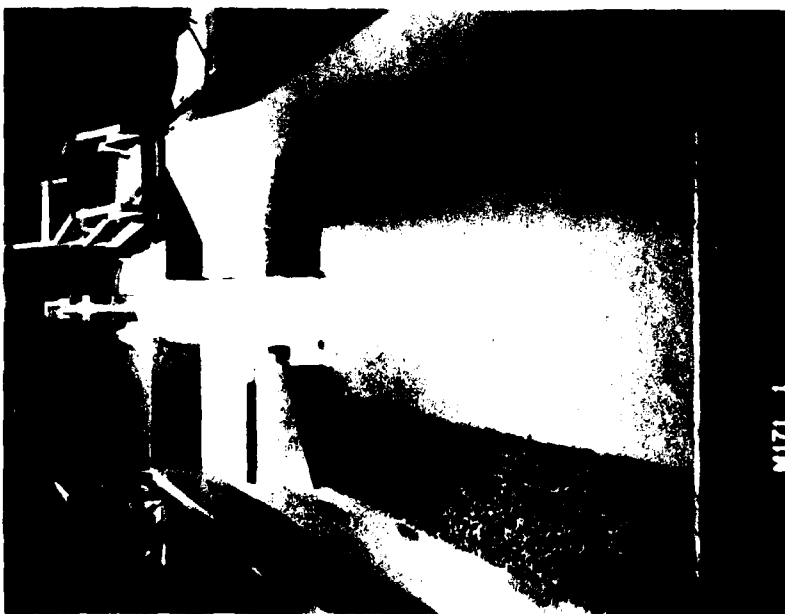
Original design spillway and plunge pool.



Photograph 5. Original design downstream channel.



Photograph 6. Regulating outlet intake tower and service bridge.

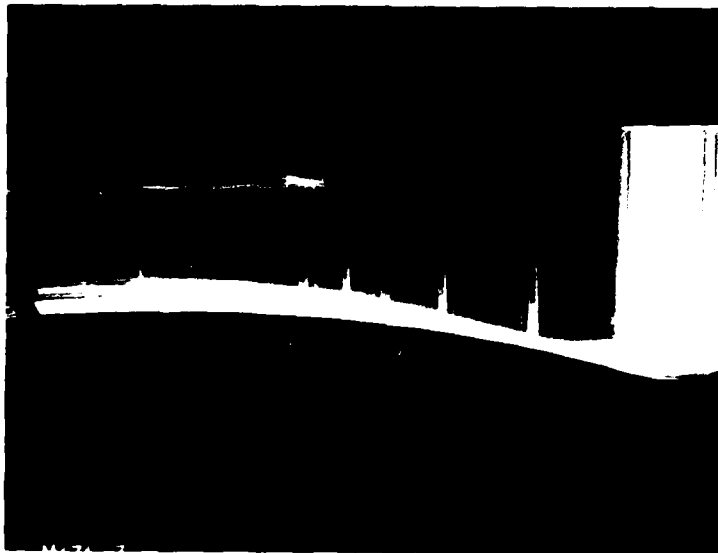


View from downstream  
Downstream channel in foreground

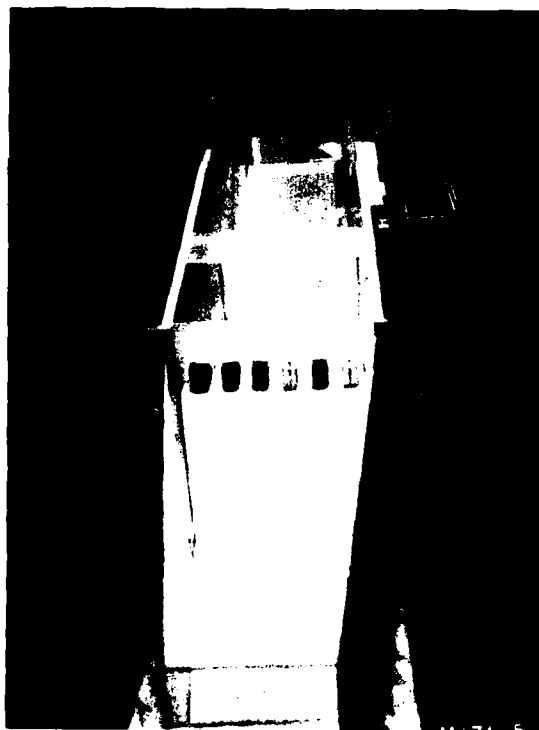


View from upstream  
Outlet tunnel in foreground

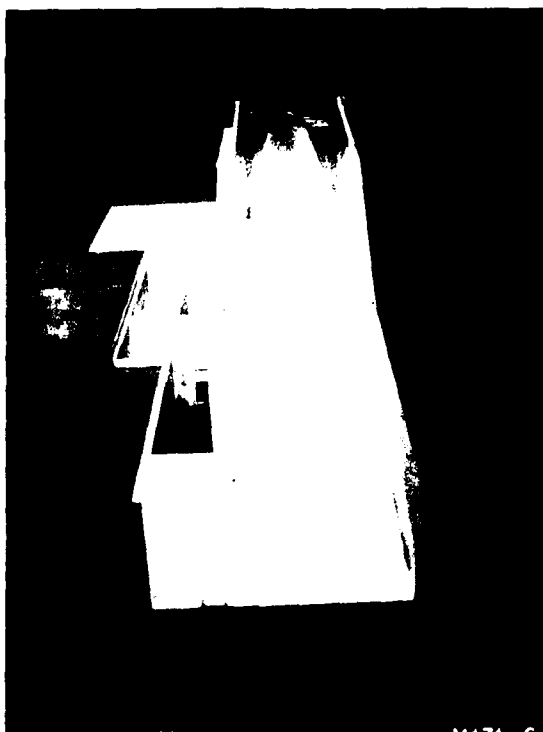
Photograph 7. Applegate regulating outlet model.



Photograph 8. Tunnel exit and chute.



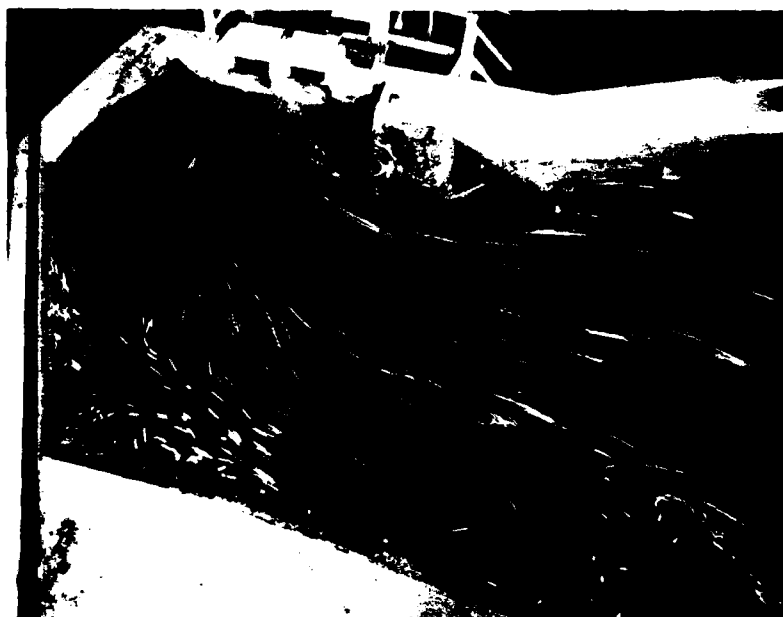
Photograph 9. Primary stilling basin.  
View from upstream.



Photograph 10. Secondary stilling basin and fishway channel and entrance. View from downstream.



Photograph 11. Stilling basins, fish barrier sill, and fishway. Overhead view.



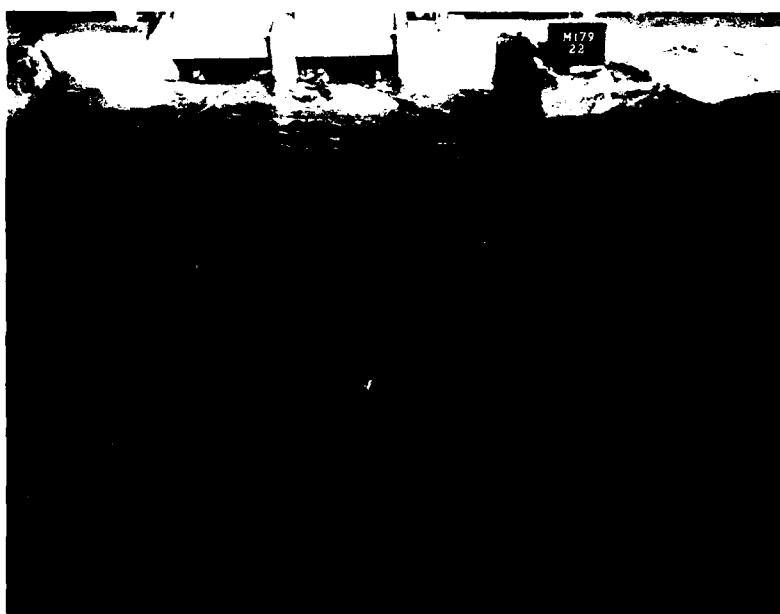
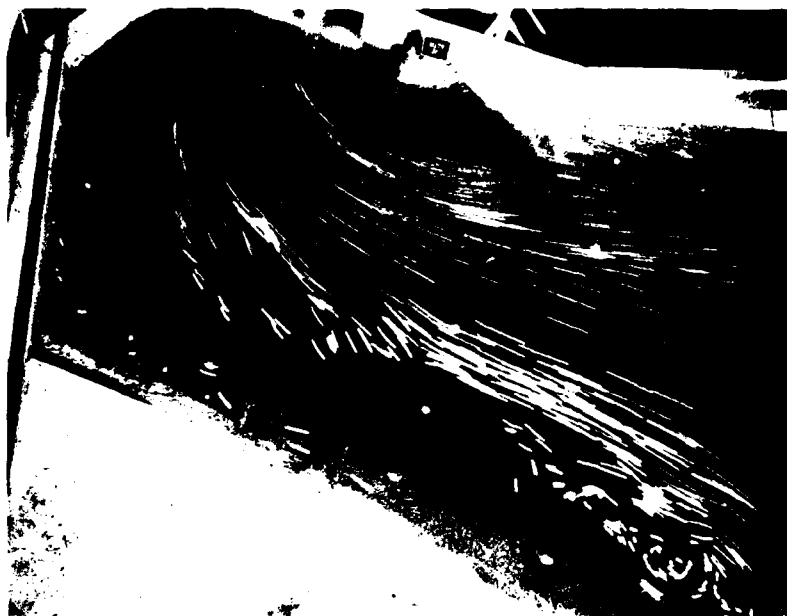
Photograph 12. Original design spillway approach channel,  
elevation 1,890,  $p/H_d = 1.24$ . Spillway  
discharge 93,600 cfs. Pool elevation 1,985.



Photograph 13. Spillway approach channel at elevation 1,900,  $p/H_d = 1.01$ . Spillway discharge 93,600 cfs. Pool elevation 1,985.



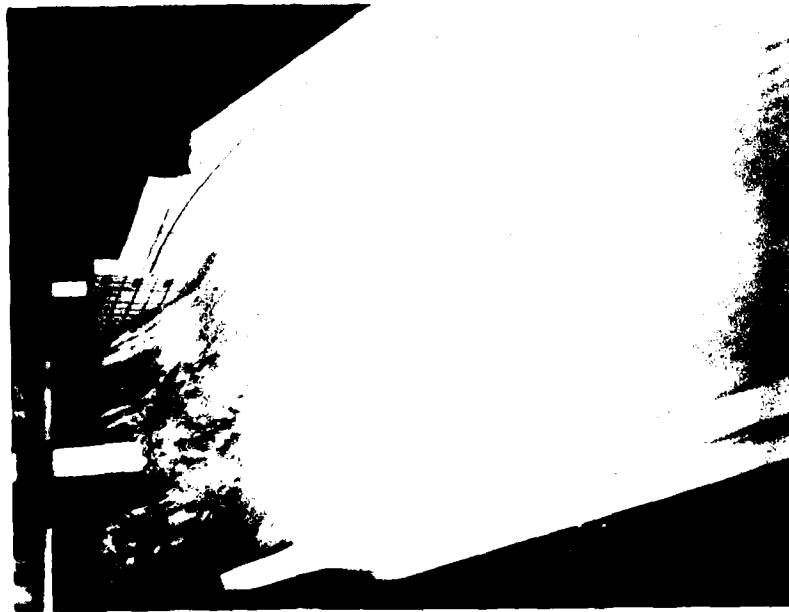
Photograph 14. Final design spillway and approach channel at elevation 1,910,  $p/H_d = 0.78$ . Spillway discharge 93,600 cfs. Pool elevation 1,985.



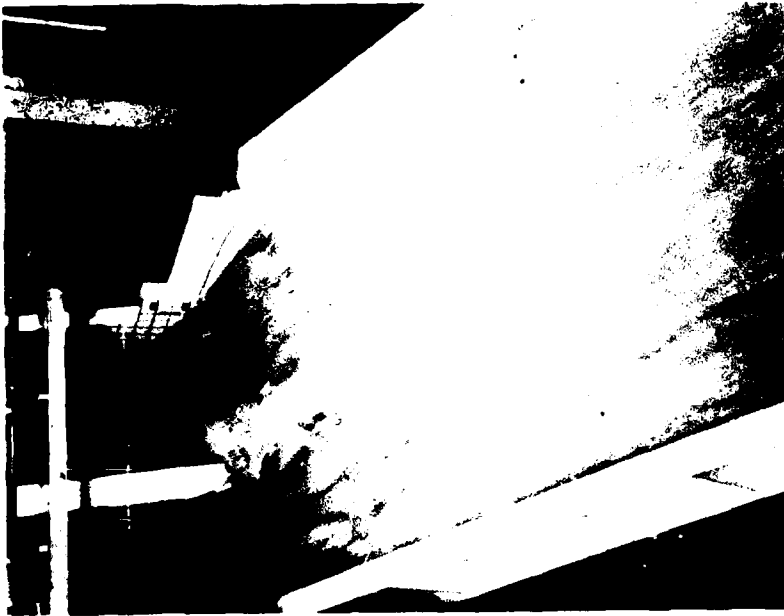
Photograph 15. Right bank abutment fill removed to elevation 1,970. Spillway approach channel at elevation 1,910,  $p/H_d = 0.78$ . Spillway discharge 93,600 cfs. Pool elevation 1,985.



Photograph 16. Final design spillway approach channel  
at elevation 1,910,  $p/H_d = 0.78$ .

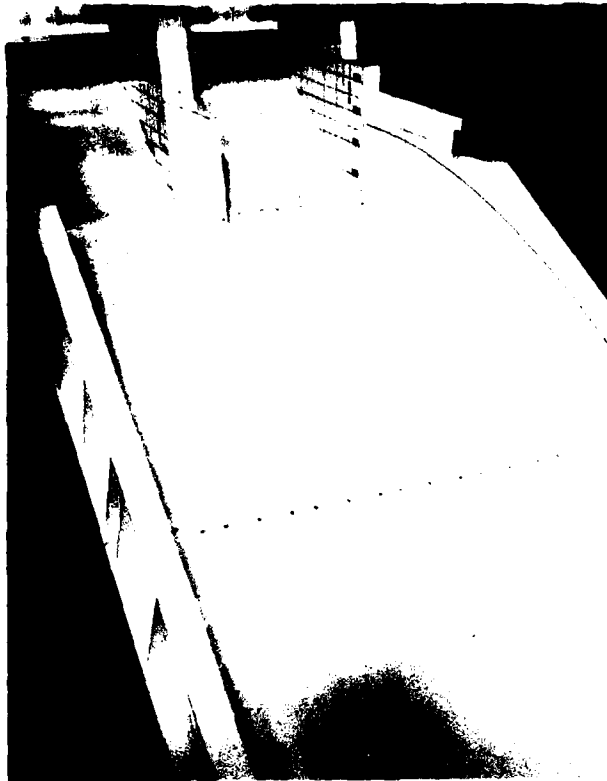


a. Discharge 43,900 cfs (SPF).

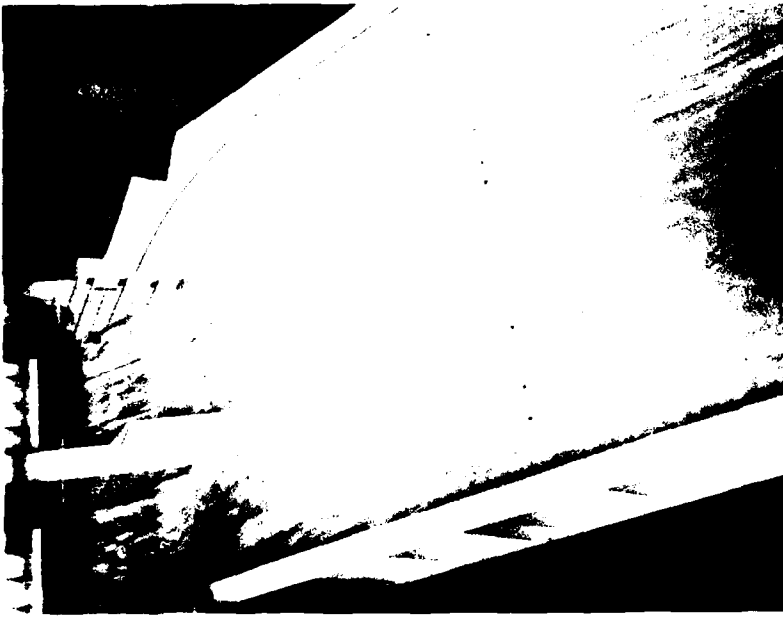


b. Discharge 93,600 cfs (SDF).

Photograph 17. Flow conditions without pier extension, original design.



Photograph 18. Spillway pier extension.



a. Discharge 43,900 cfs (SPF).

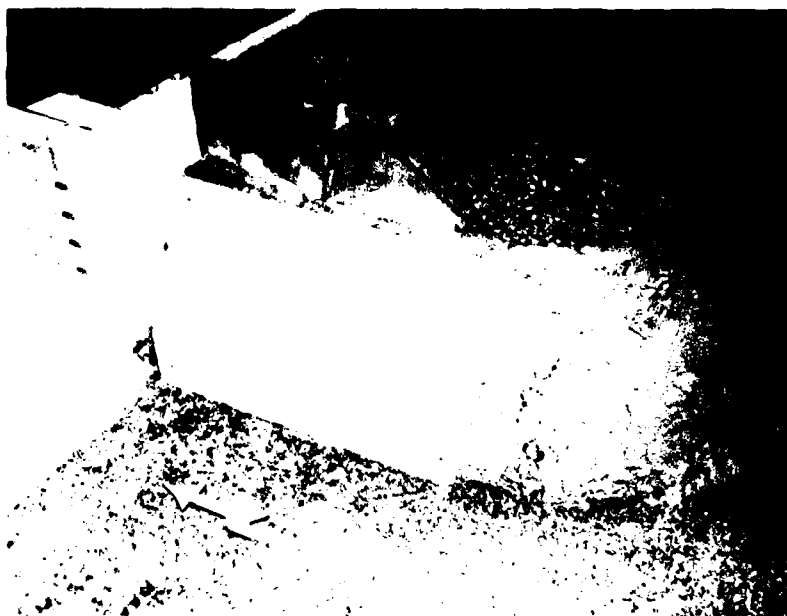


b. Discharge 93,600 cfs (SDF).

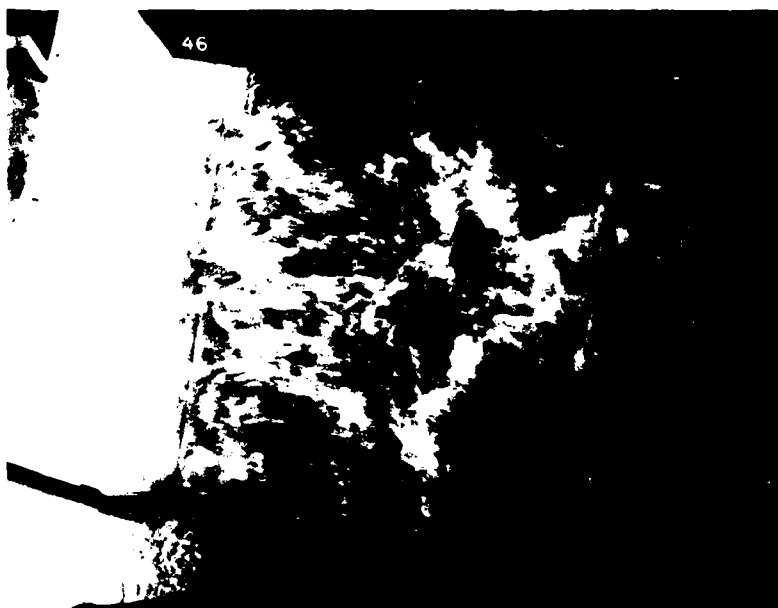
Photograph 19. Flow condition with pier extension.



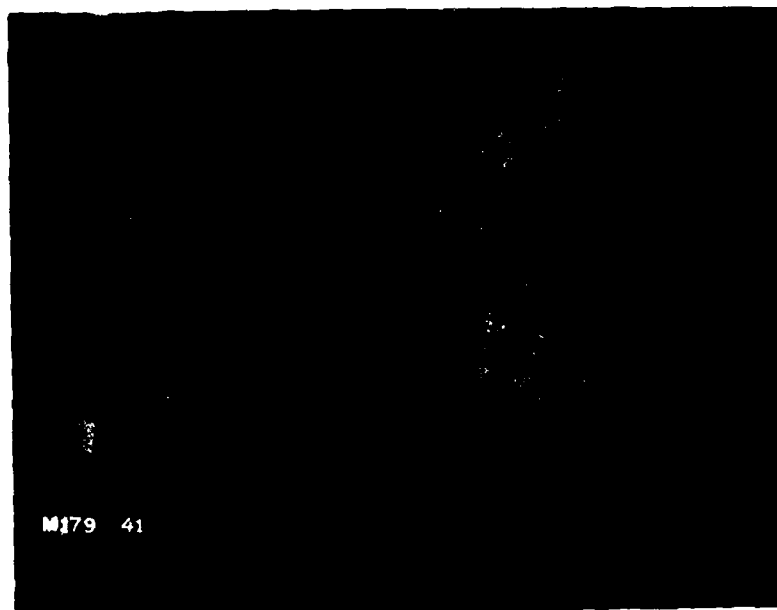
Photograph 20. Vertical wall extending upstream from right spillway abutment.



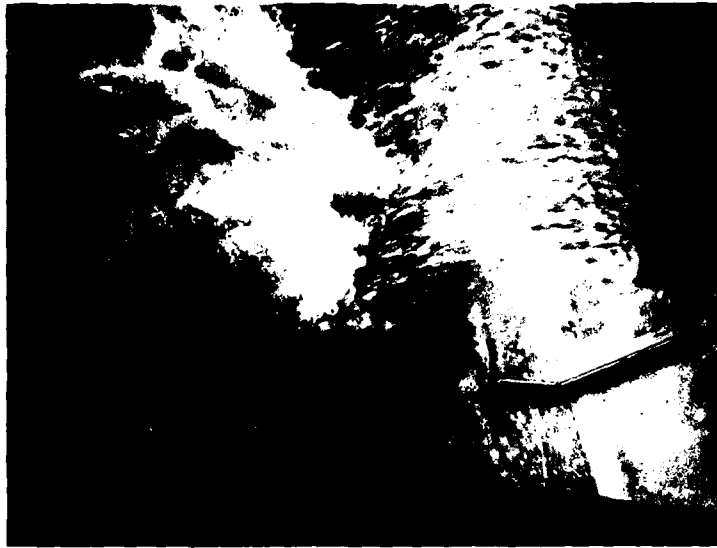
Photograph 21. Concrete stabilizing blanket on right bank of spillway approach channel, top at elevation 1,970.



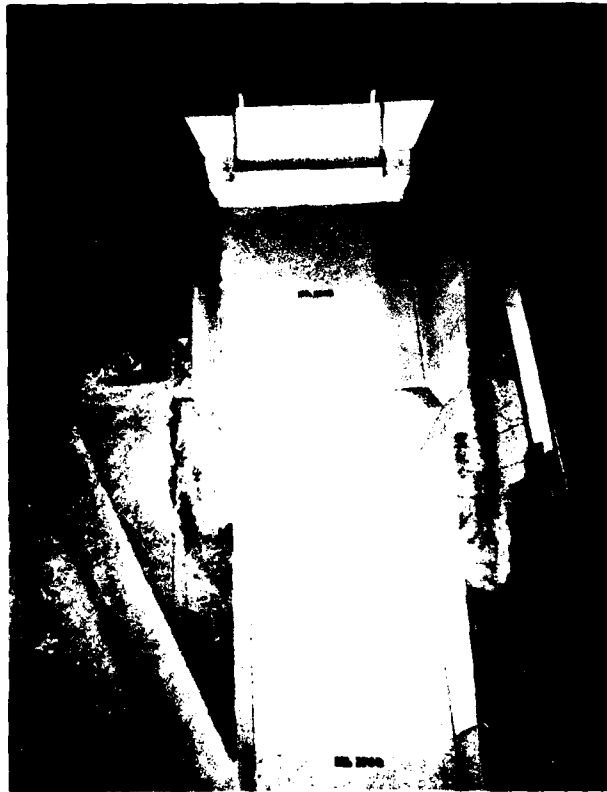
Photograph 22. Flow from lower flip bucket. Spillway discharge 7,000 cfs.



Photograph 23. Flow from upper flip bucket. Spillway discharge 7,000 cfs.



Photograph 24. Flow from upper flip bucket. Spillway discharge 4,500 cfs.



Photograph 25. Original design plunge pool and downstream channel.



Photograph 26a.



Photograph 26b.

Flow from upper flip bucket. Spillway discharge 18,400 cfs.



Photograph 26c. Original design plunge pool and downstream channel. Spillway discharge 18,400 cfs.



Photograph 26d. Left bank.



Photograph 26e. Right bank.

Original design plunge pool and downstream channel.  
Spillway discharge 18,400 cfs.

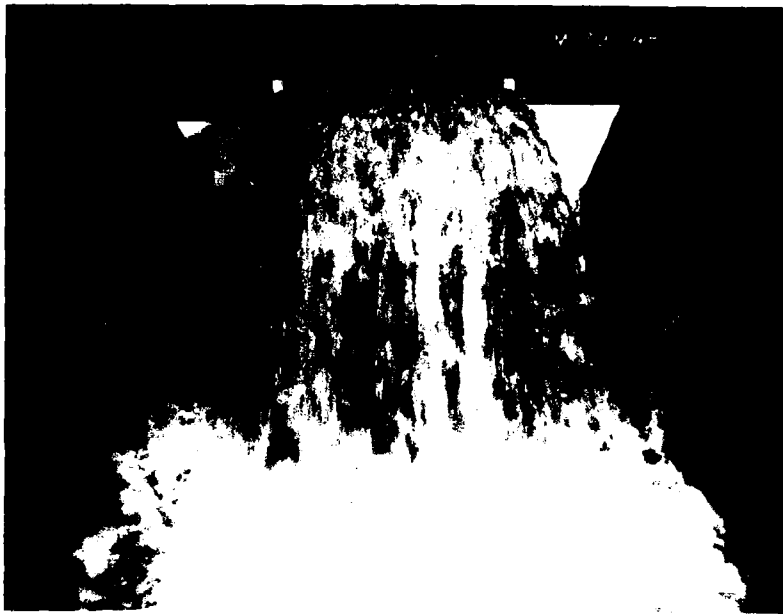


Photograph 27a.



Photograph 27b.

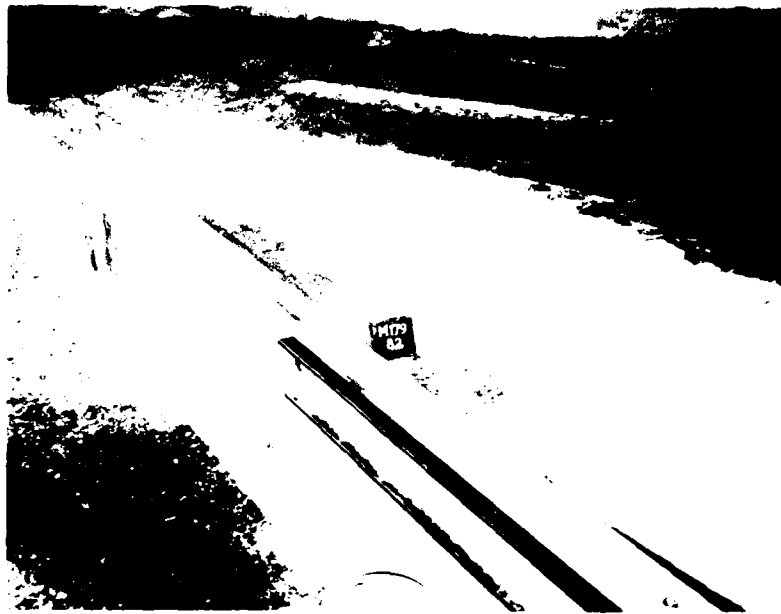
Flow from upper flip bucket.  
Spillway discharge 29,700 cfs.



Photograph 27c. Flow from upper flip bucket. Spillway discharge 29,700 cfs.



Photograph 27d. Original design plunge pool and downstream channel. Spillway discharge 29,700 cfs.

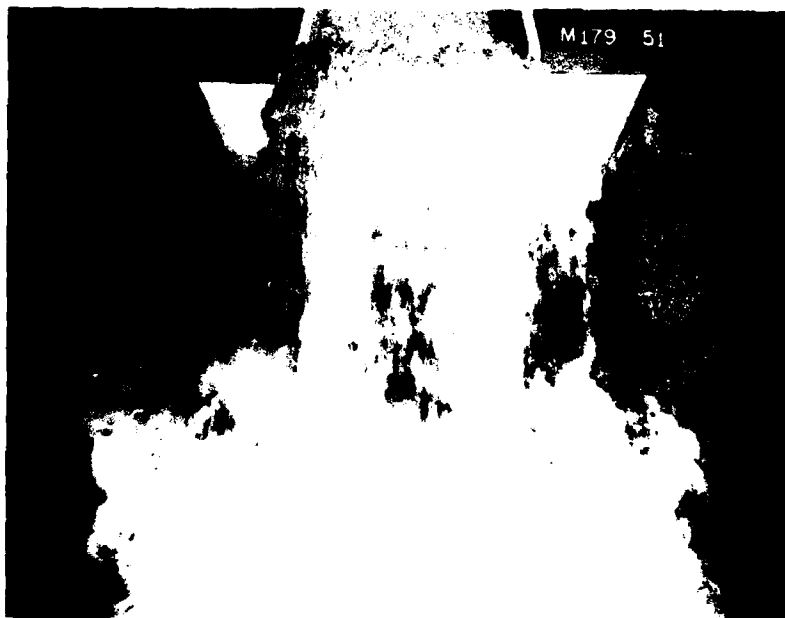


Photograph 27e. Left bank.



Photograph 27f. Right bank.

Original design plunge pool and downstream channel.  
Spillway discharge 29,700 cfs.



Photograph 28a. Flow from upper flip bucket. Spillway discharge 43,900 cfs.



Photograph 28b.

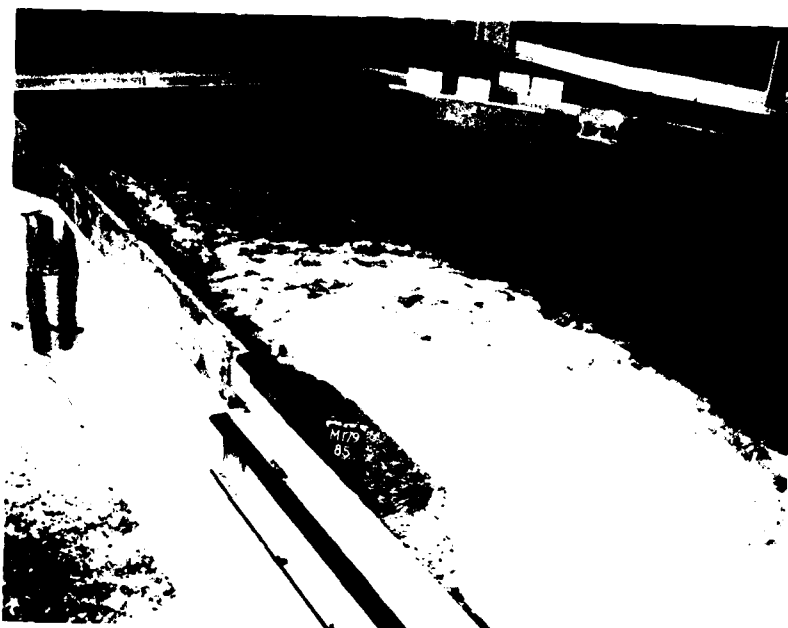


Photograph 28c.

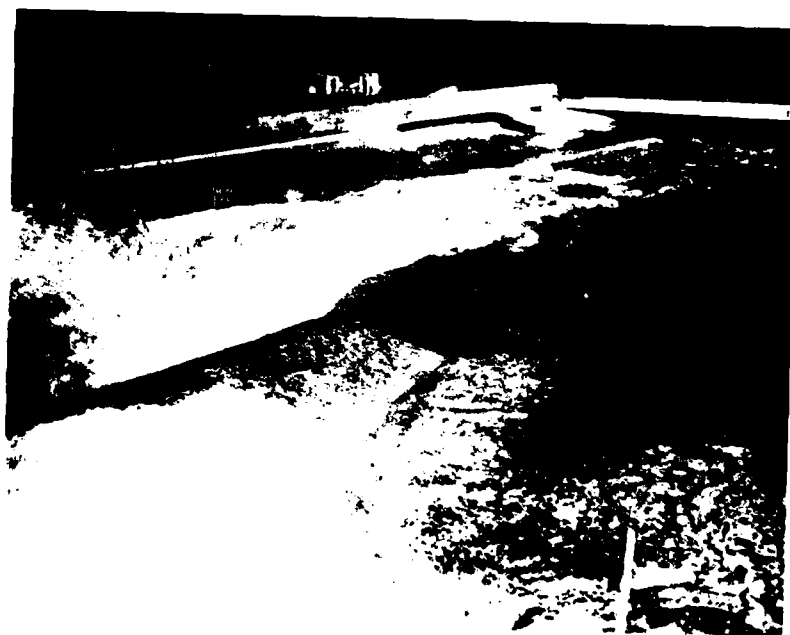
Flow from upper flip bucket. Spillway discharge 43,900 cfs.



Photograph 28d. Original design plunge pool and downstream channel. Spillway discharge 43,900 cfs.



Photograph 28e. Left bank.

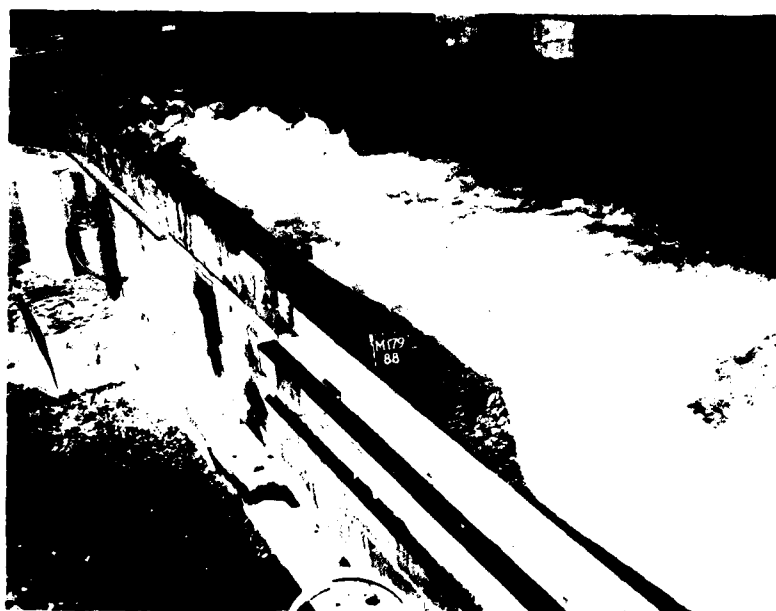


Photograph 28f. Right bank.

Original design plunge pool and downstream channel.  
Spillway discharge 43,900 cfs.



Photograph 29a. Original design plunge pool and downstream channel. Spillway discharge 93,600 cfs.

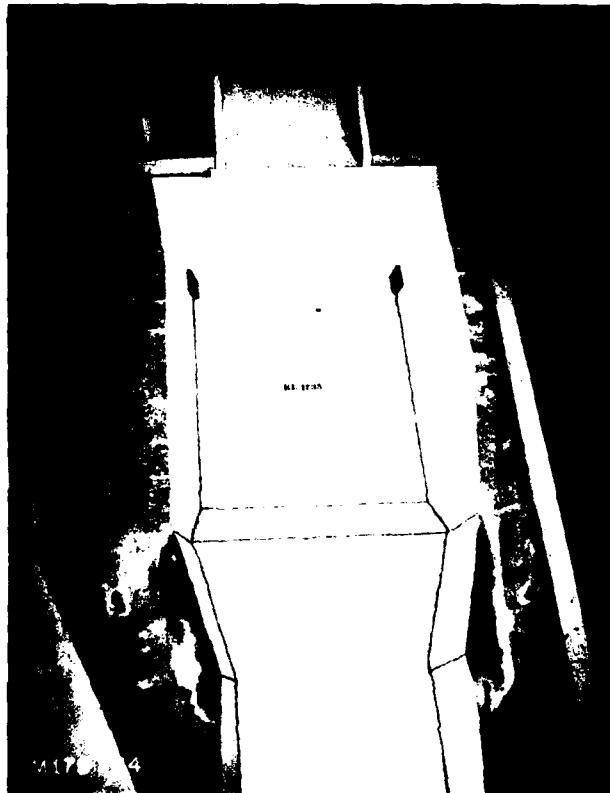


Photograph 29b. Left bank.

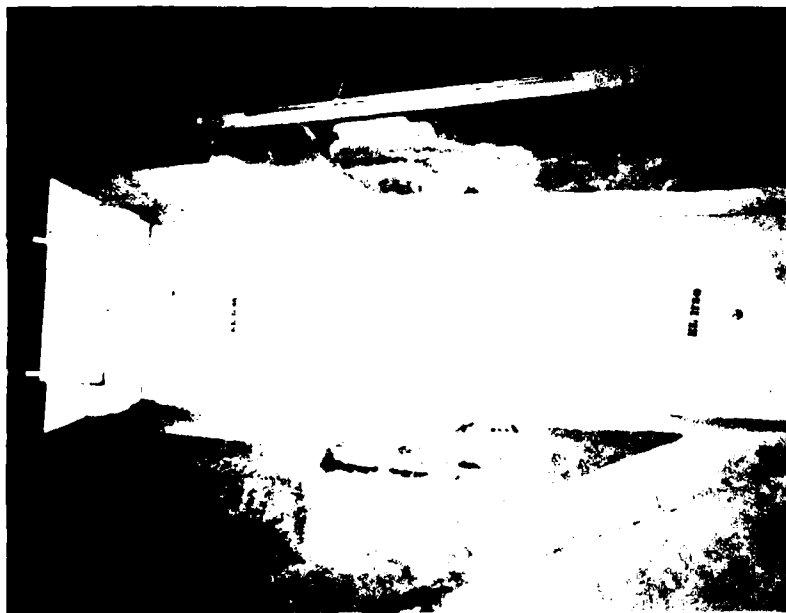


Photograph 29c. Right bank.

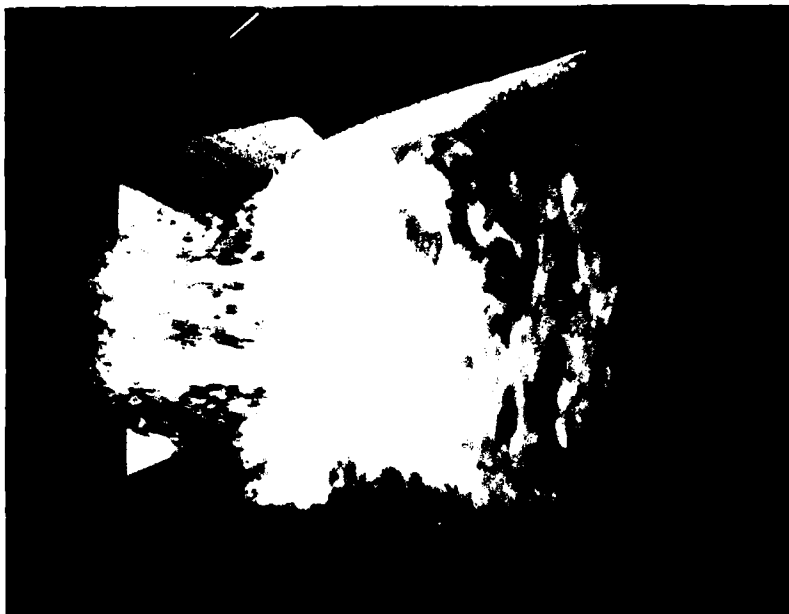
Original design plunge pool and downstream channel.  
Spillway discharge 93,600 cfs.



Photograph 30. Revised left and right bank transition downstream from plunge pool.



Photograph 31. Plan B plunge pool.  
Pool length extended  
80 feet.



Photograph 32a. Plan B plunge pool.  
Spillway discharge  
29,700 cfs.



Photograph 32b. Left side.

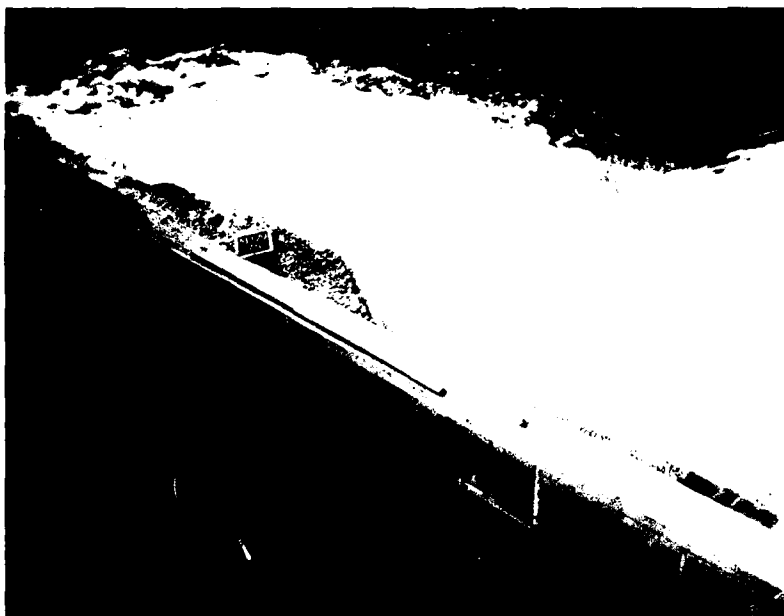


Photograph 32c. Right side.

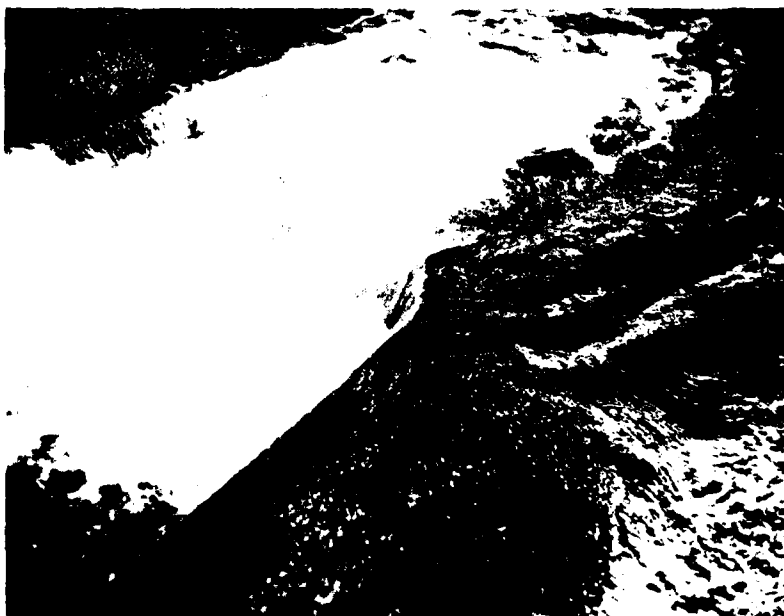
Plan B plunge pool.  
Spillway discharge 29,700 cfs.



Photograph 33a. Plan B plunge pool. Spillway discharge 43,900 cfs.

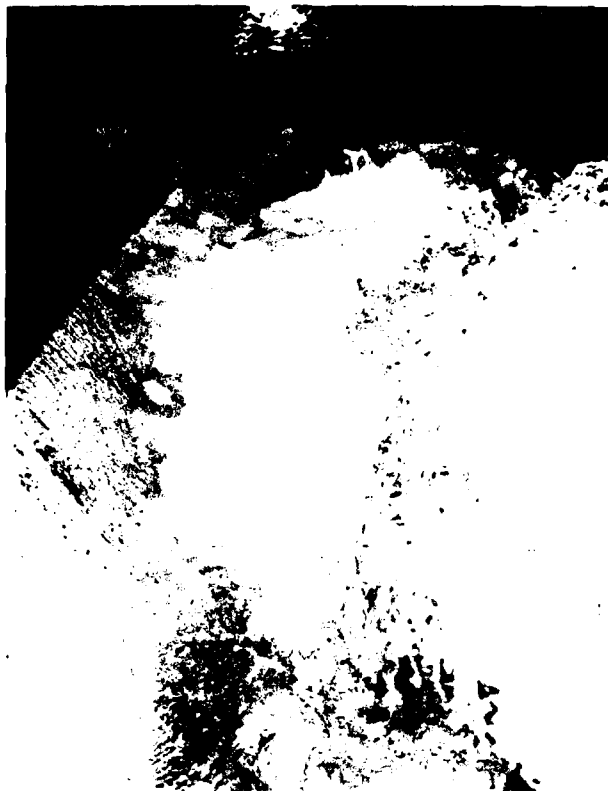


Photograph 33b. Left side.



Photograph 33c. Right side.

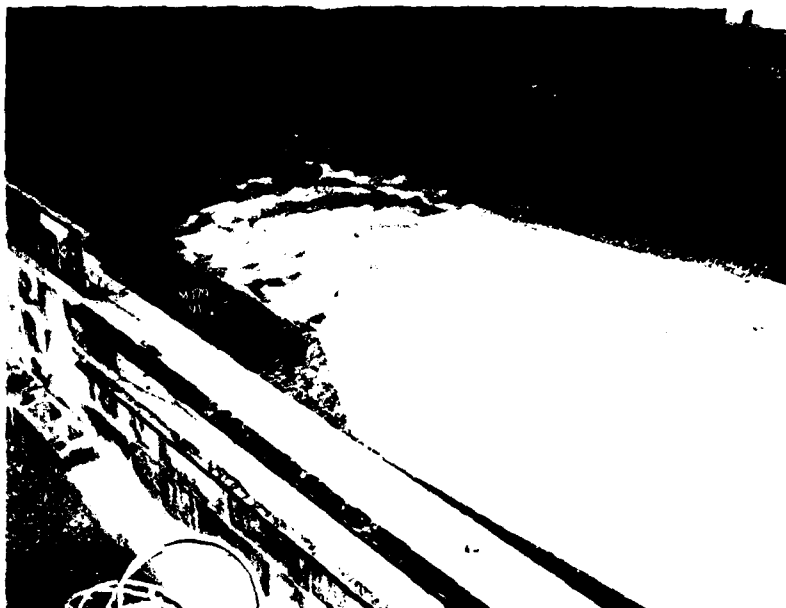
Plan B plunge pool.  
Spillway discharge 43,900 cfs.



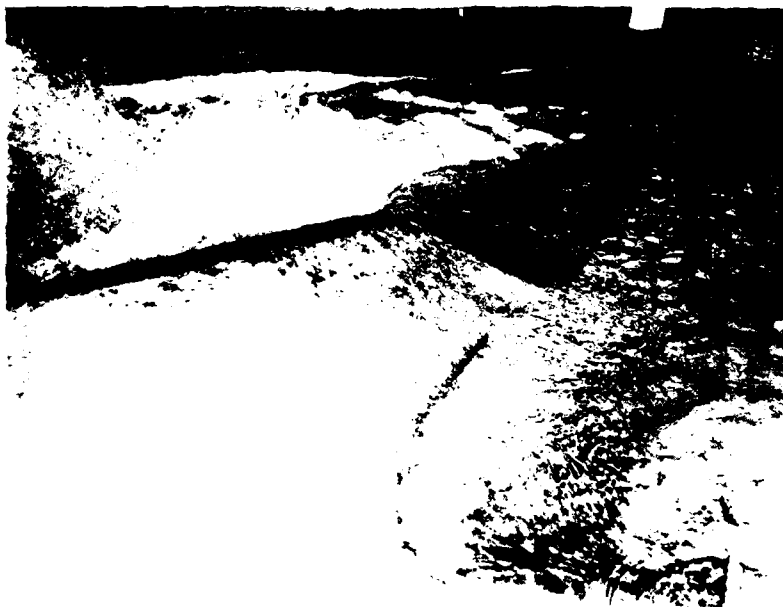
Photograph 34a. Upwelling along left bank. Original design plunge pool and downstream channel. Spillway discharge 29,700 cfs. Right bay only operating.



Photograph 34b. Original design plunge pool and downstream channel. Spillway discharge 29,700 cfs. Right bay only operating.



Left bank



Right bank

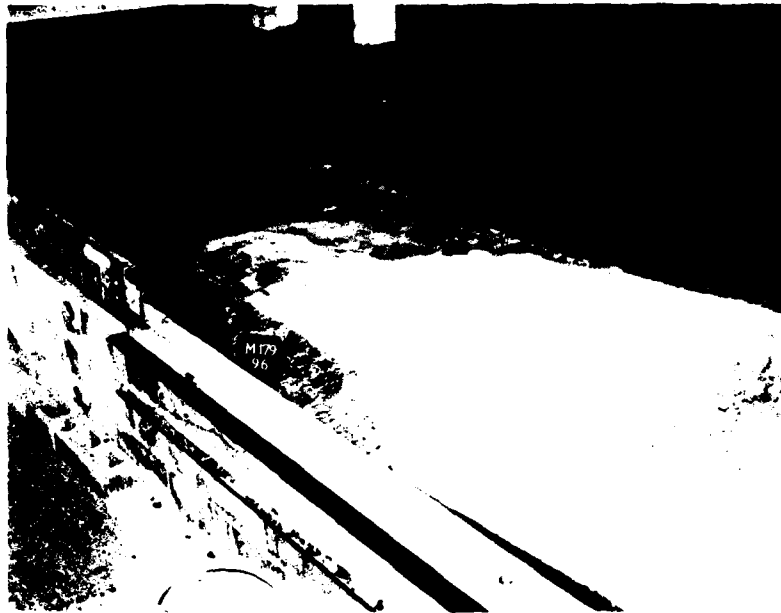
Photograph 35. Original design plunge pool and downstream channel. Spillway discharge 29,700 cfs. Right bay only operating.



Photograph 36a. Plan B plunge pool and original downstream channel. Spillway discharge 29,700 cfs. Right bay only operating.



Photograph 36b. Upwelling along left bank. Plan B  
plunge pool and original downstream  
channel. Spillway discharge 29,700 cfs.  
Right bay only operating.

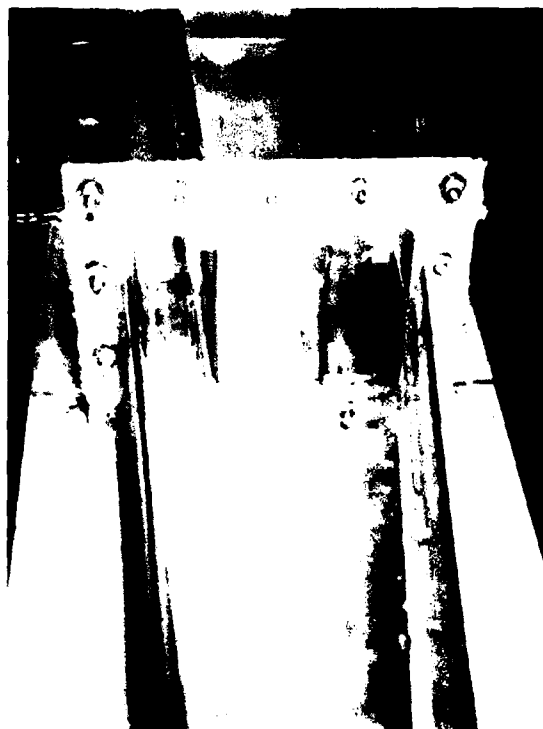


Left bank

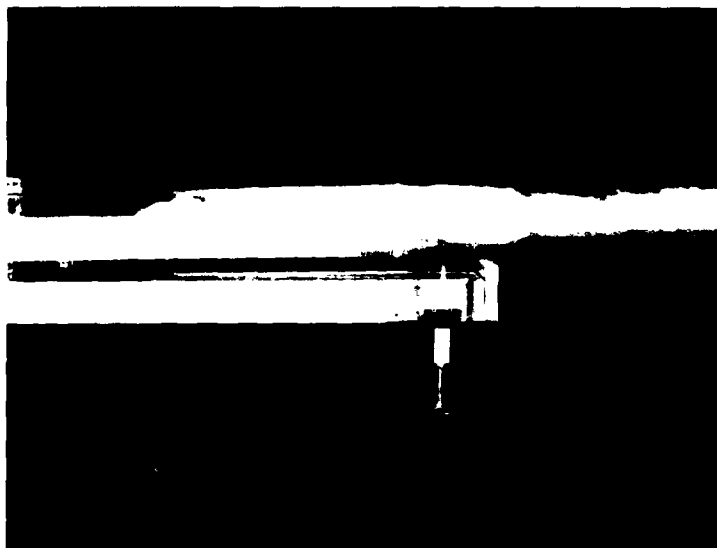


Right bank

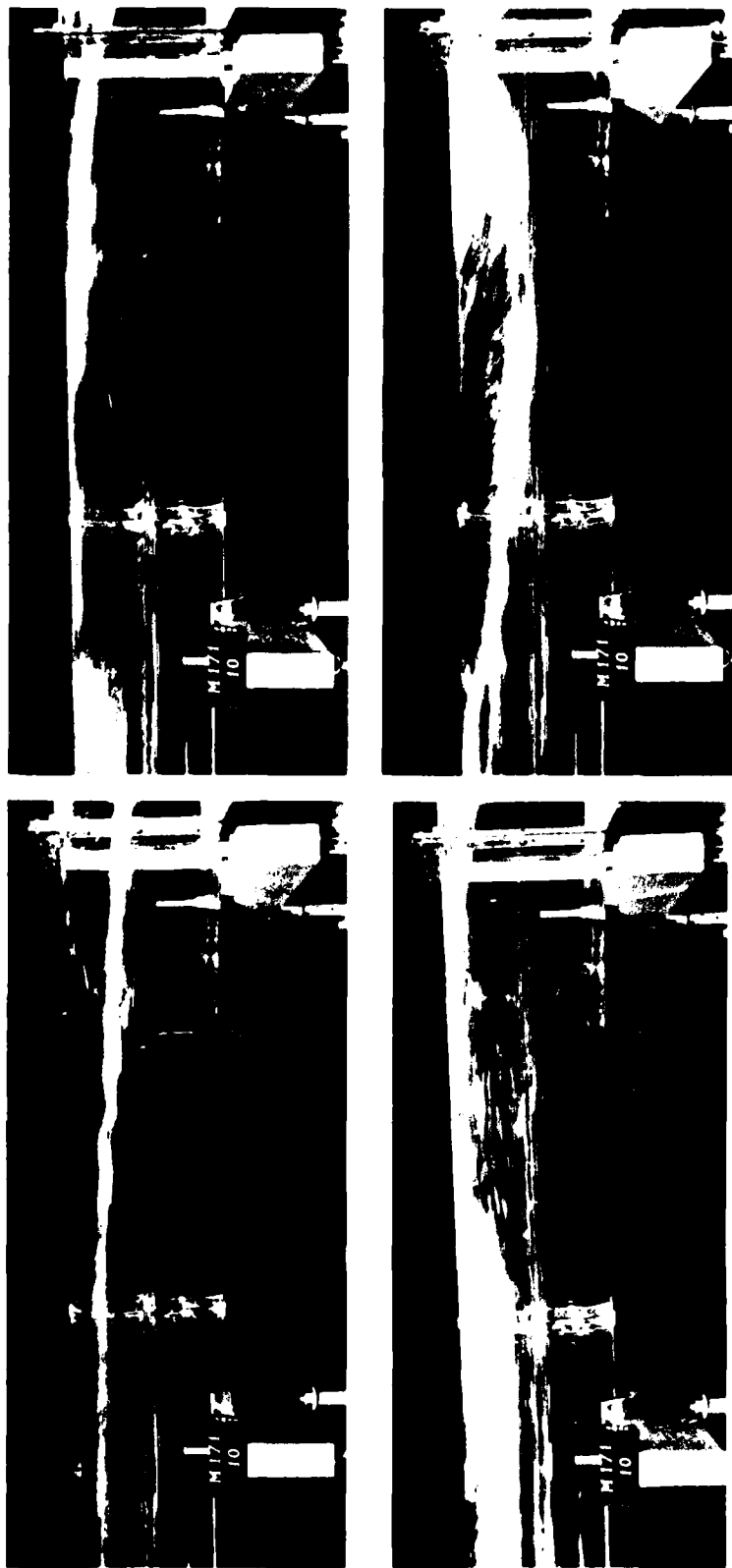
Photograph 37. Plan B plunge pool and original downstream channel. Spillway discharge 29,700 cfs. Right bay only operating.



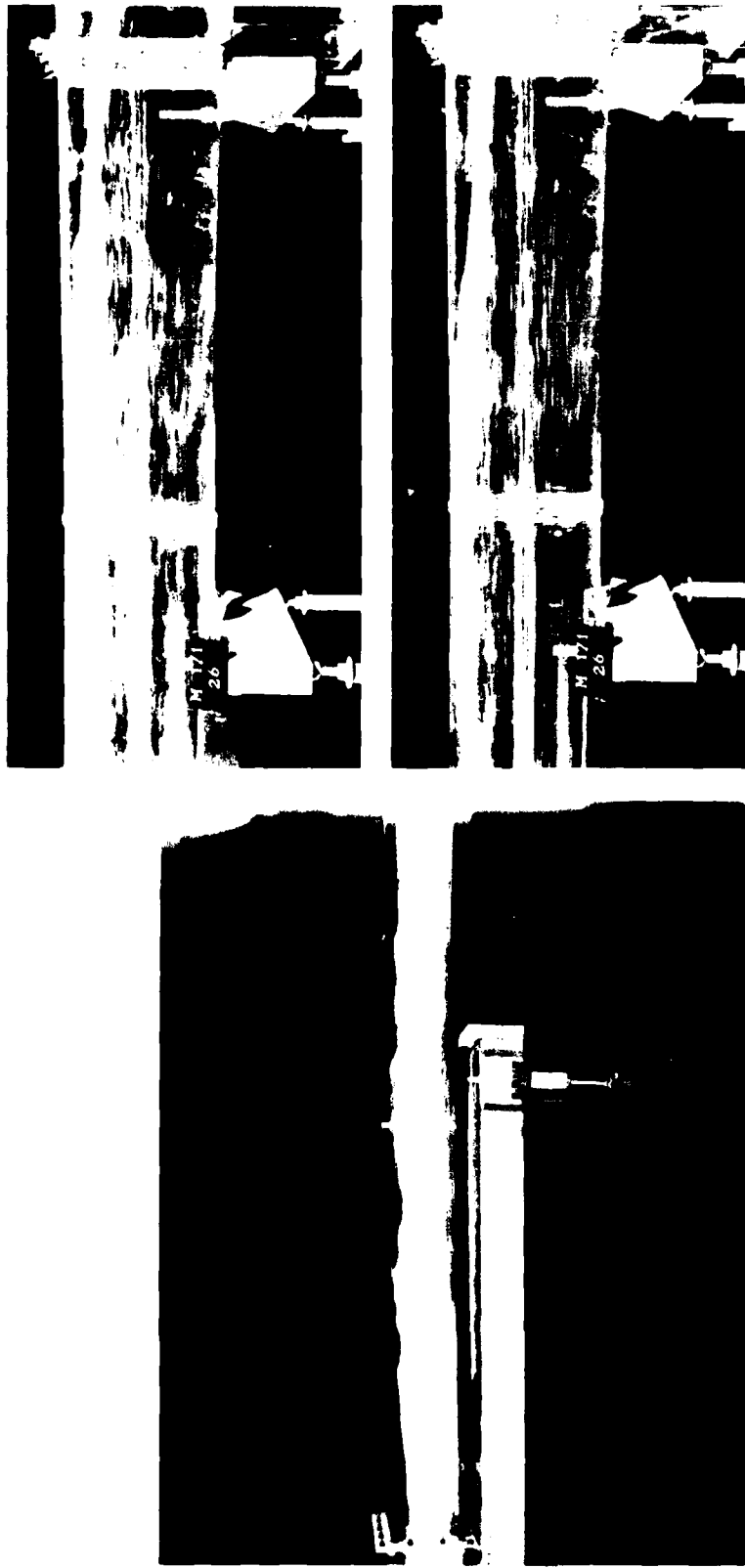
Photograph 38. Downstream end of splitter wall.



Photograph 39. Rooster tail at upstream end of tunnel.  
Original design splitter wall. Discharge  
6,100 cfs.



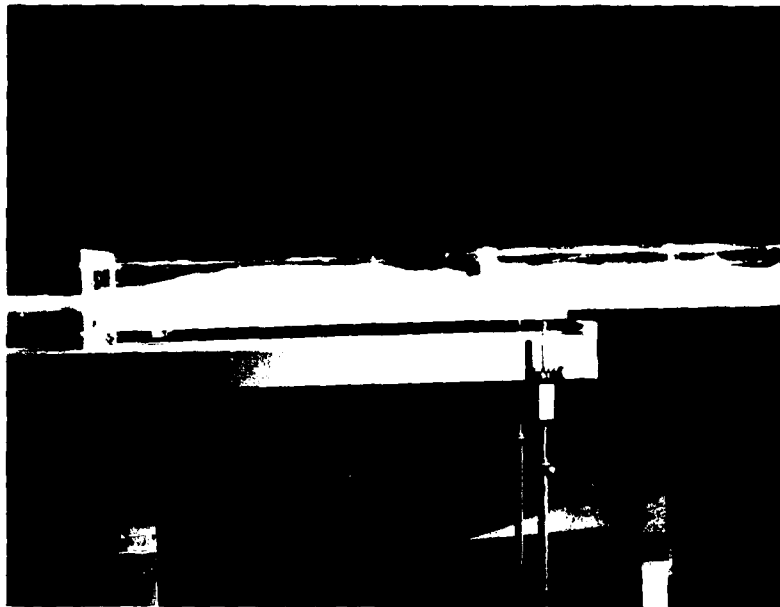
Photograph 40. Variable flow depth near downstream end of tunnel.  
Original design splitter wall. Discharge 6,100 cfs.



Rooster tail at upstream end of tunnel.

Variable flow depth near downstream  
end of tunnel.

Photograph 41. Flow with splitter wall plan 8. Discharge 6,100 cfs.

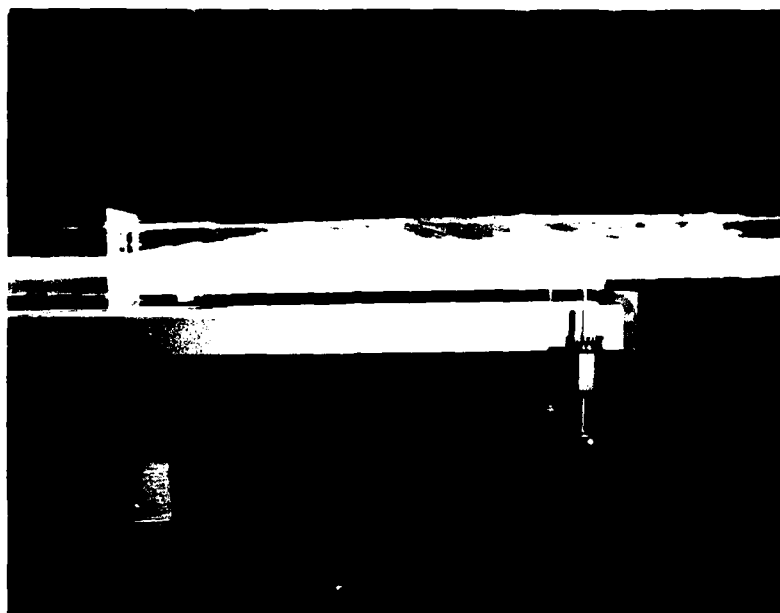


Upstream end of tunnel.

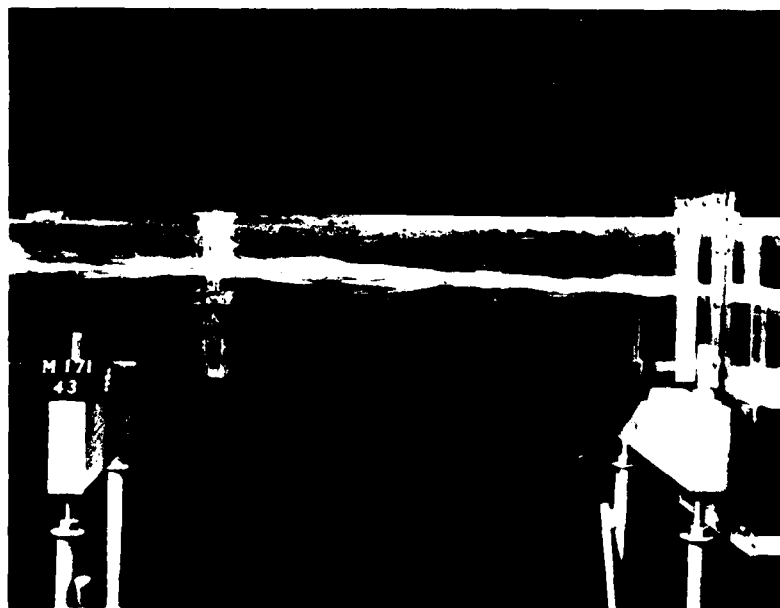


Near downstream end of tunnel.

Photograph 42. Flow with splitter wall plan 8. Discharge 4,500 cfs, pool elevation 1,889.

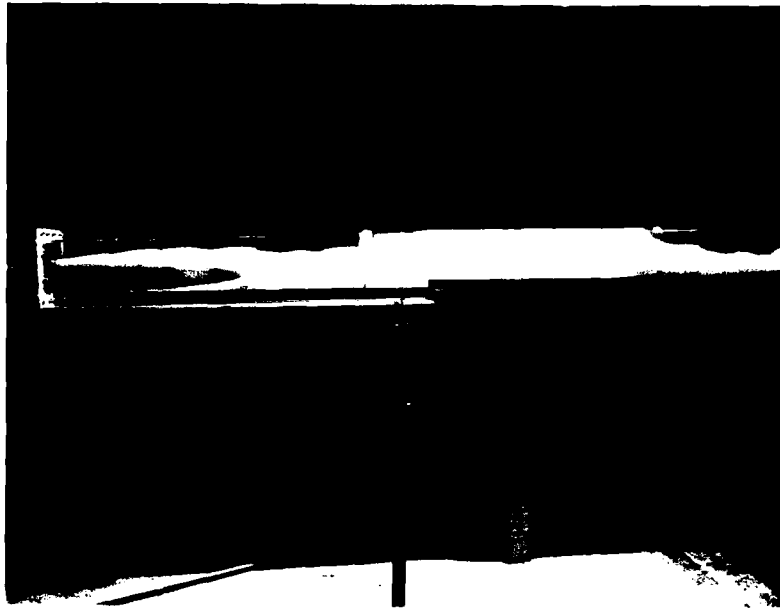


Upstream end of tunnel.

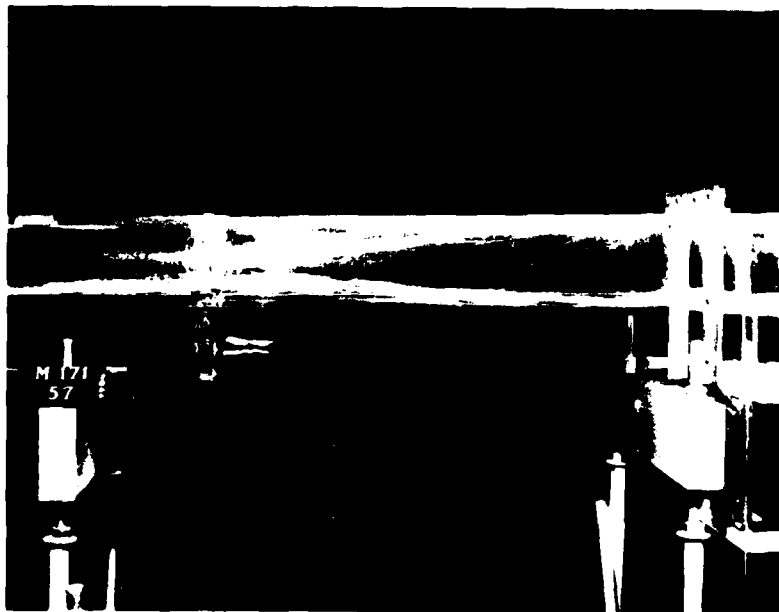


Near downstream end of tunnel.

Photograph 43. Flow with splitter wall plan 8. Discharge 4,500 cfs, pool elevation 1,987.



Upstream end of tunnel.

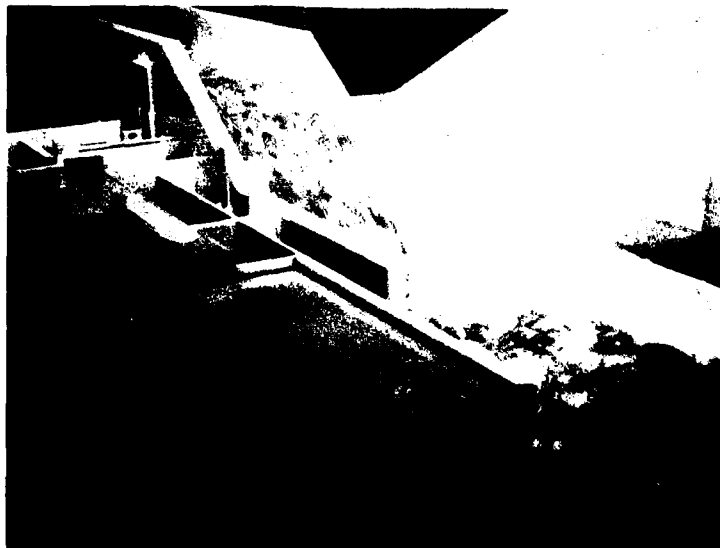


Near downstream end of tunnel.

Photograph 44. Flow with splitter wall plan 8. Discharge 3,050 cfs, pool elevation 1,987. Left valve fully open, right valve closed.



View from upstream.



View from downstream.

Photograph 45. Flow in stilling basins. Discharge  
6,100 cfs, conduit roughness  
 $k_s = 0.002$ .

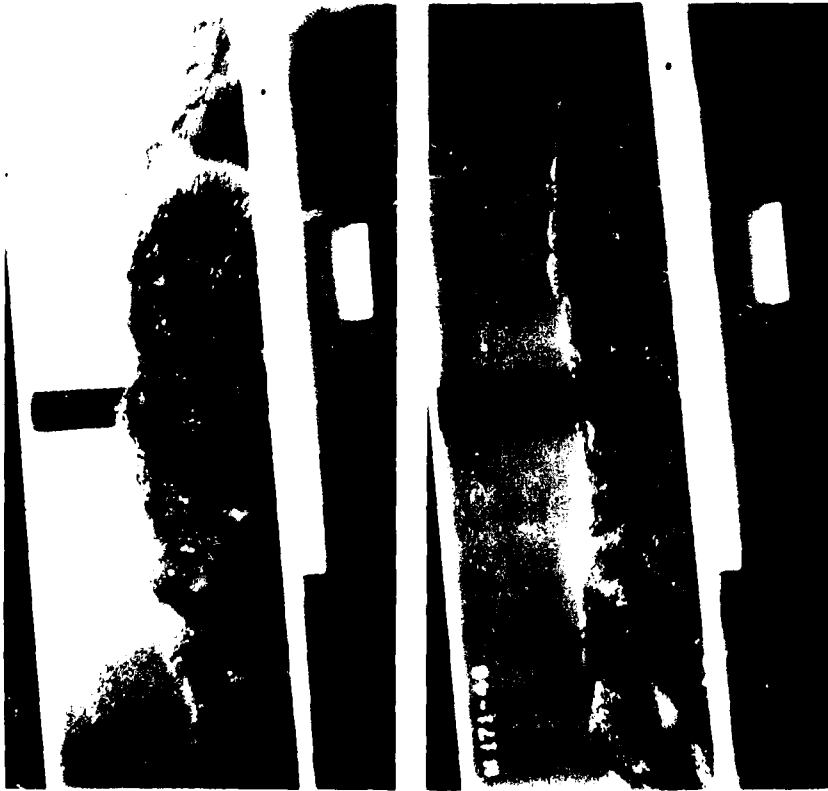


View from upstream.



View from downstream.

Photograph 46. Flow in stilling basins. Discharge  
6,100 cfs, conduit roughness  
 $k_s = 0.0002$ .



Primary stilling basin.



View from downstream.

Photograph 47. Flow in stilling basins. Discharge 4,500 cfs, pool elevation 1,987.



Primary stilling basin.

View from downstream.

Photograph 48. Flow in stilling basins. Discharge 4,500 cfs, pool elevation 1,889.



Primary stilling basin.



View from downstream.

Photograph 49. Flow in stilling basins. Discharge  
3,050 cfs, pool elevation 1,987.



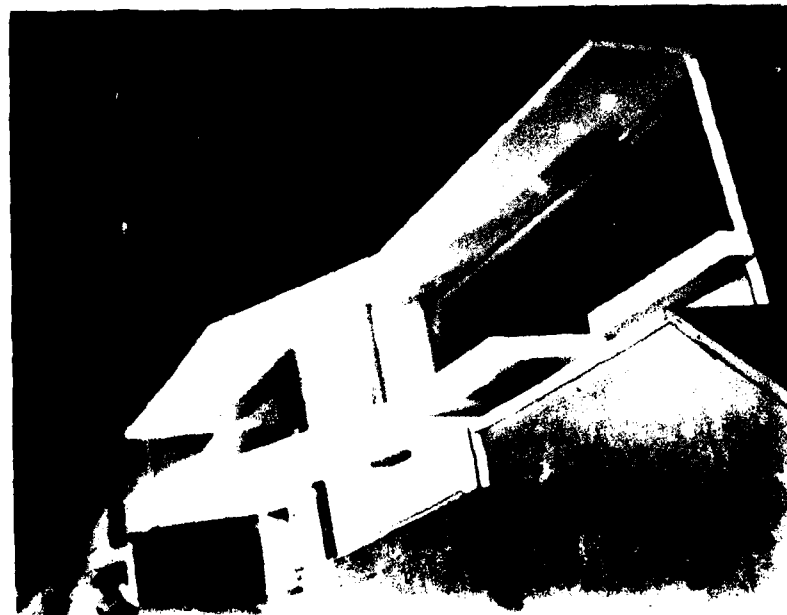
Photograph 50. Flow conditions in downstream channel  
with an outlet discharge of 6,100 cfs.



Photograph 51. 60-foot-wide channel,  
dry bed.



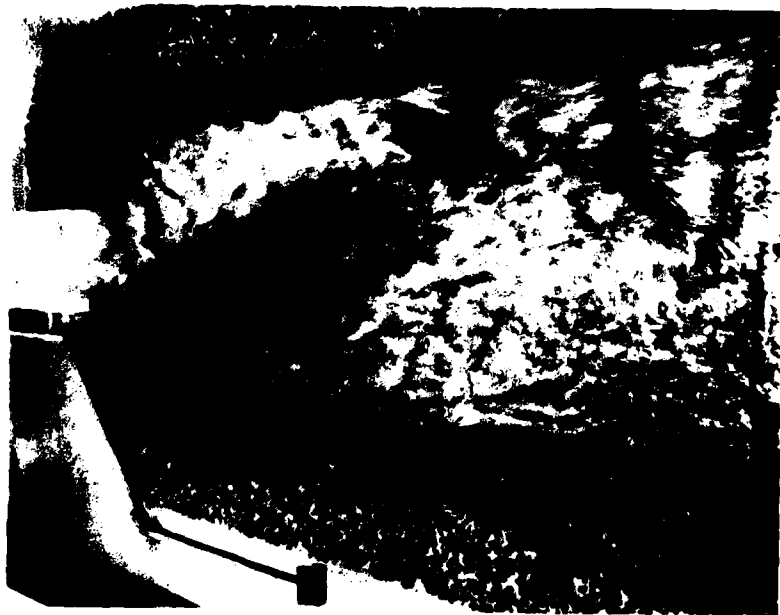
Photograph 52. Flow condition in  
60-foot-wide channel  
with outlet discharge  
of 6,100 cfs.



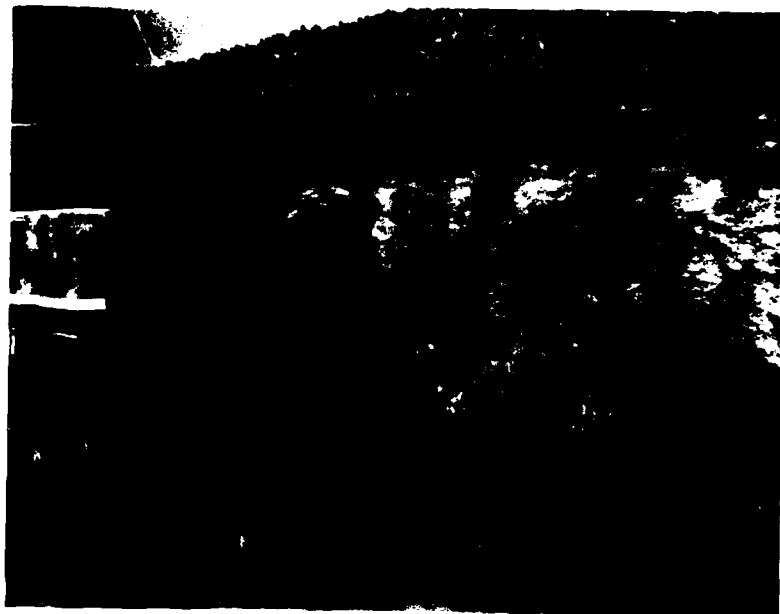
Photograph 53. Secondary basin at elevation 1,756.



Photograph 54. Secondary basin wall raised to elevation 1,770.



Photograph 55. Flow conditions with  
6,100 cfs, wall  
elevation 1,770 and  
80-foot wide channel.



Photograph 56. Flow conditions with  
6,100 cfs, wall  
elevation 1,770 and  
60-foot wide channel.



Photograph 59. Flow in plunge pool, discharge 200 cfs.  
Outlet discharge 3,050 cfs, pool  
elevation 1,987.

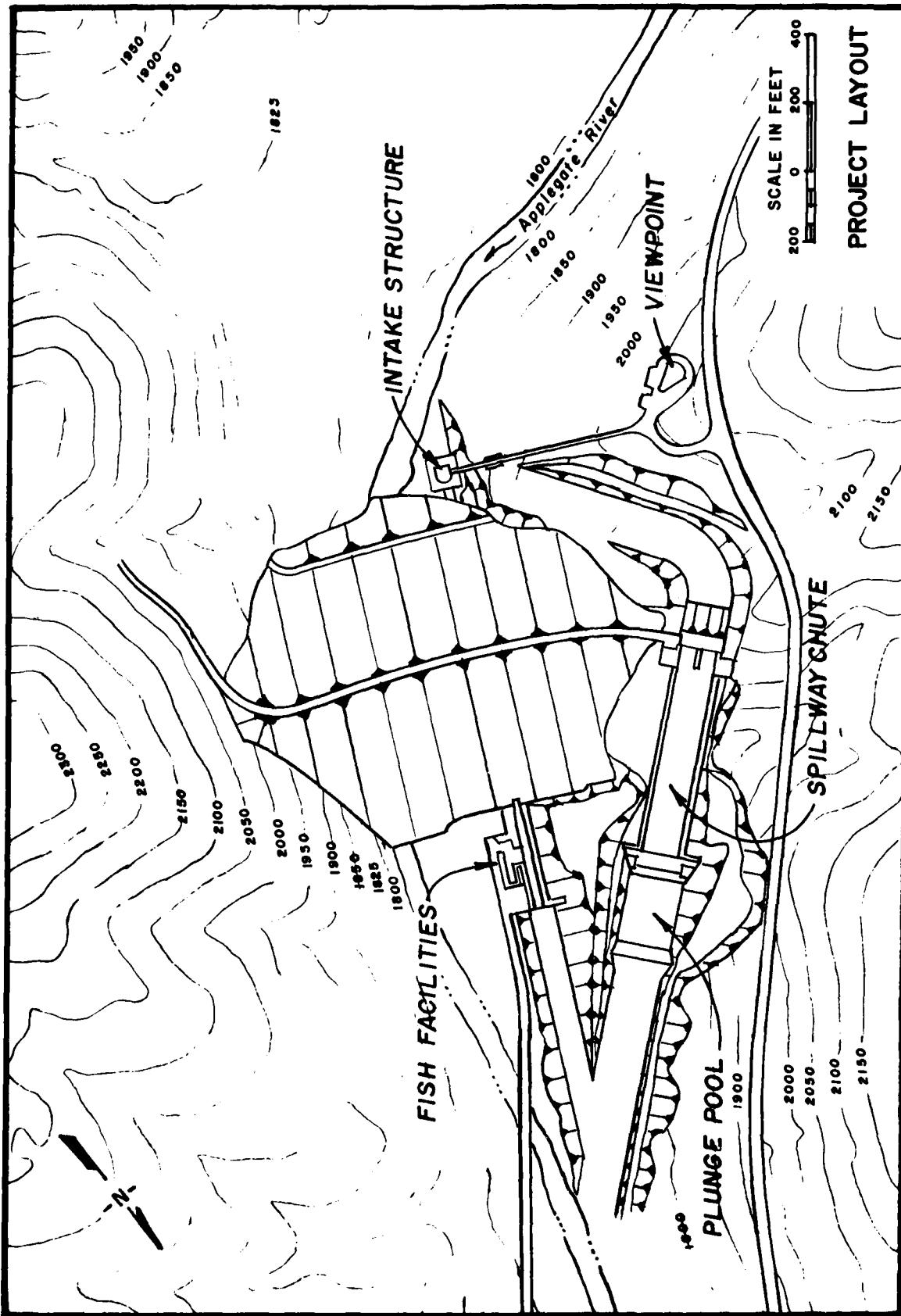
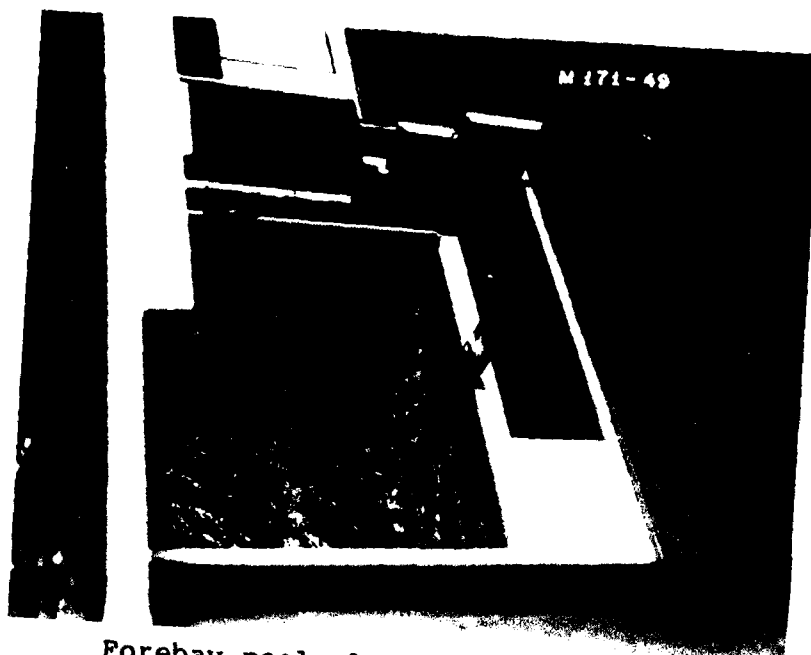


PLATE 1



Photograph 57. Flow in plunge pool, discharge  
200 cfs. Outlet discharge  
6,100 cfs.

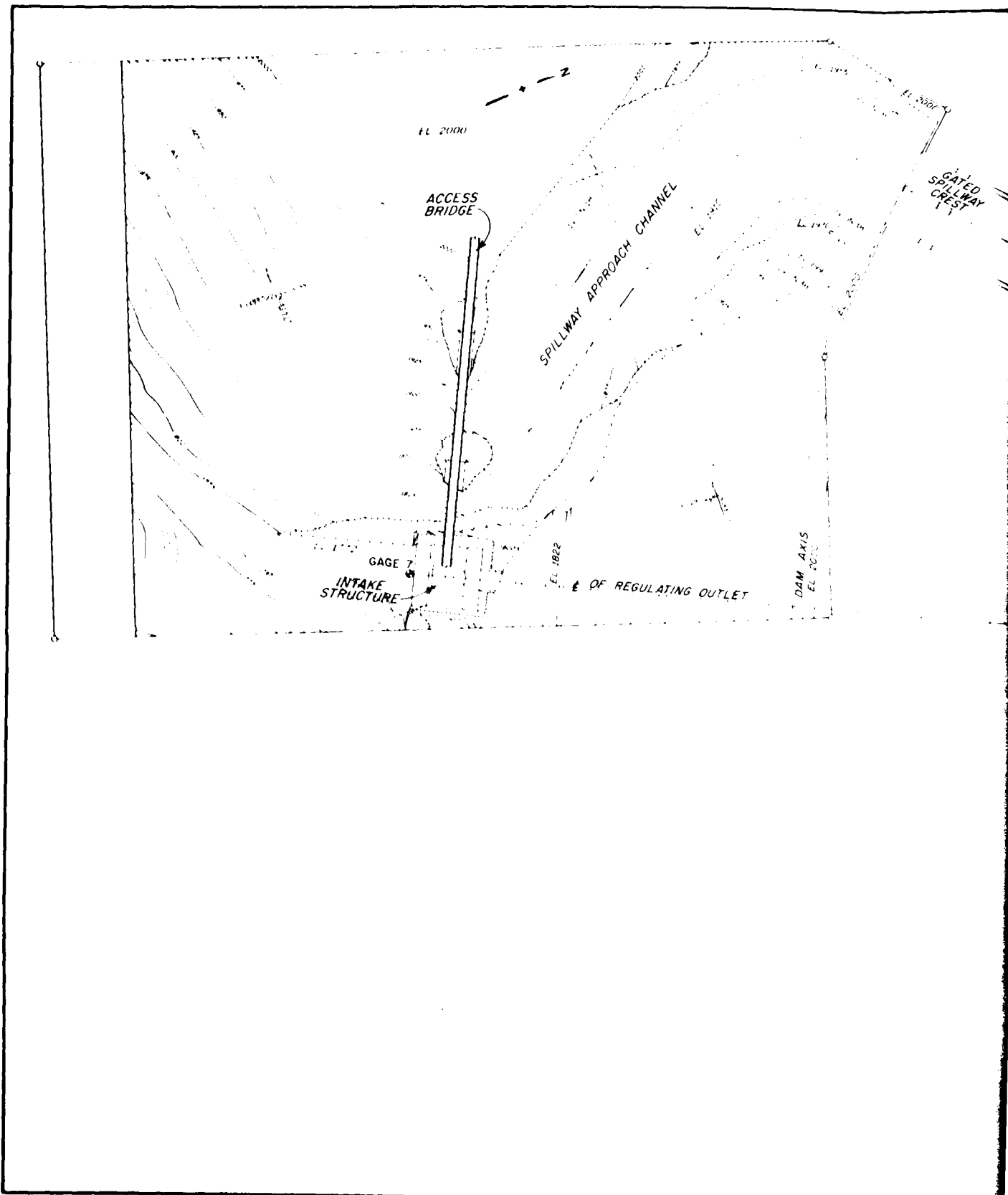


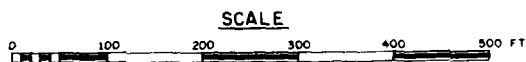
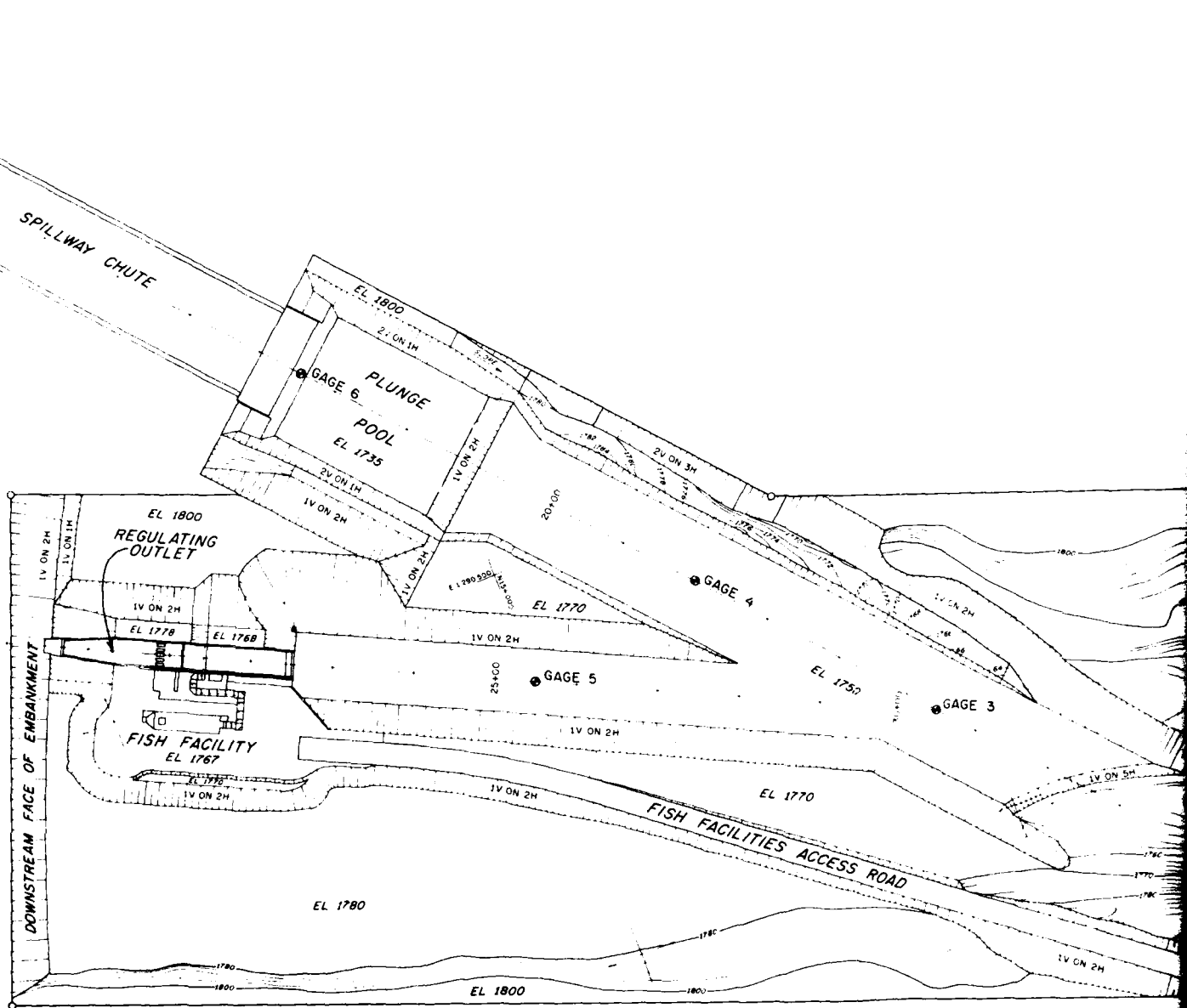
Forebay pool elevation 1,987.

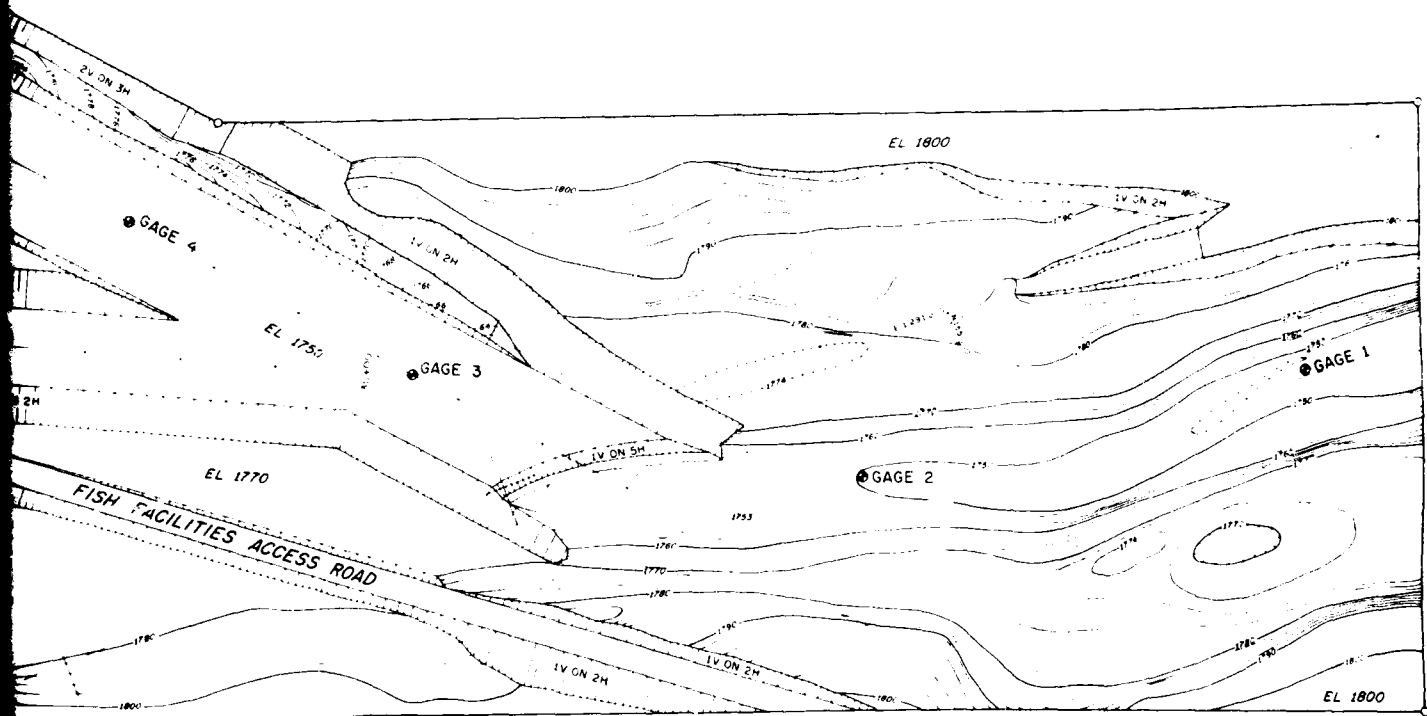


Forebay pool elevation 1,889.

Photograph 58. Flow in plunge pool, discharge  
200 cfs. Outlet discharge  
4,500 cfs.







1:30-SCALE MODEL LAYOUT

AD-A147 010

APPLEGATE DAM SPILLWAY AND OUTLET WORKS APPLEGATE RIVER  
OREGON HYDRAULIC..(U) CORPS OF ENGINEERS PORTLAND OREG  
NORTH PACIFIC DIV M M KUBO ET AL. SEP 84 TR-171-1

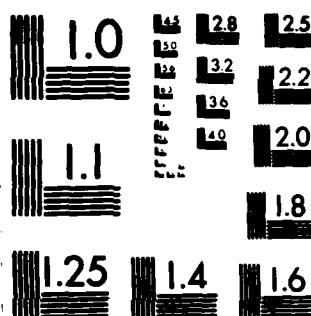
2/2

UNCLASSIFIED

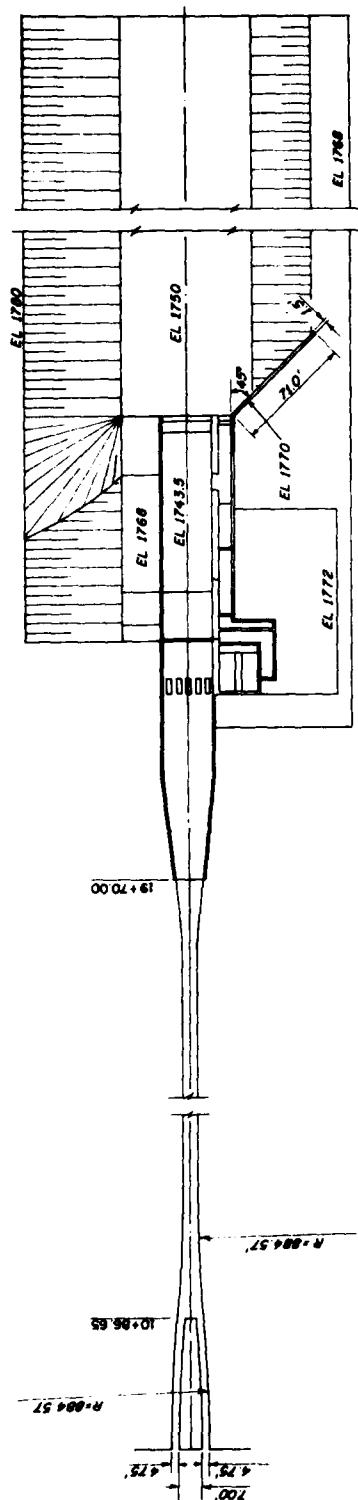
F/G 13/2

NL

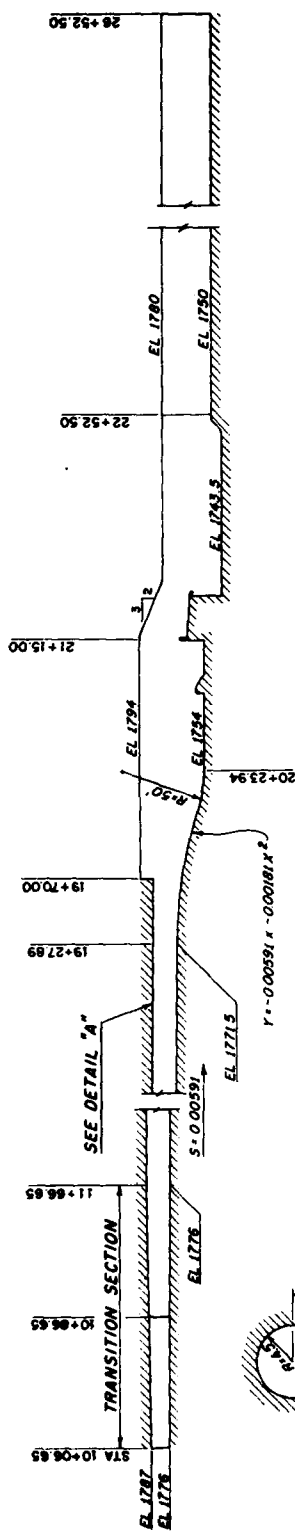

END  
DATE  
FILMED  
12-84  
DTIC



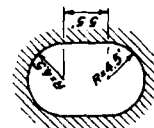
MICROCOPY RESOLUTION TEST CHART  
NATIONAL BUREAU OF STANDARDS-1963-A



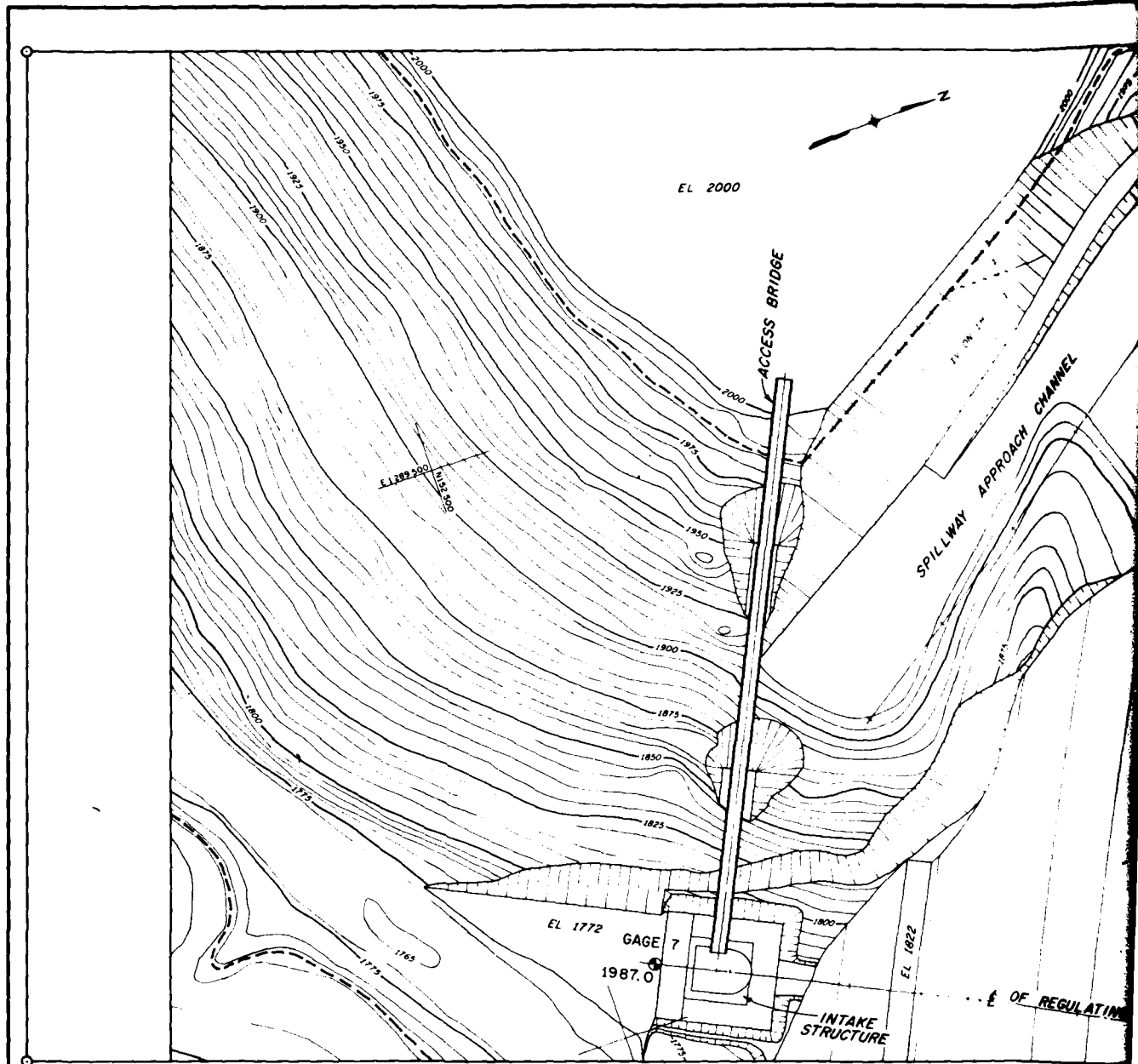
PLAN



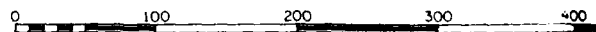
SECTION ALONG CENTER LINE



DETAIL "A"

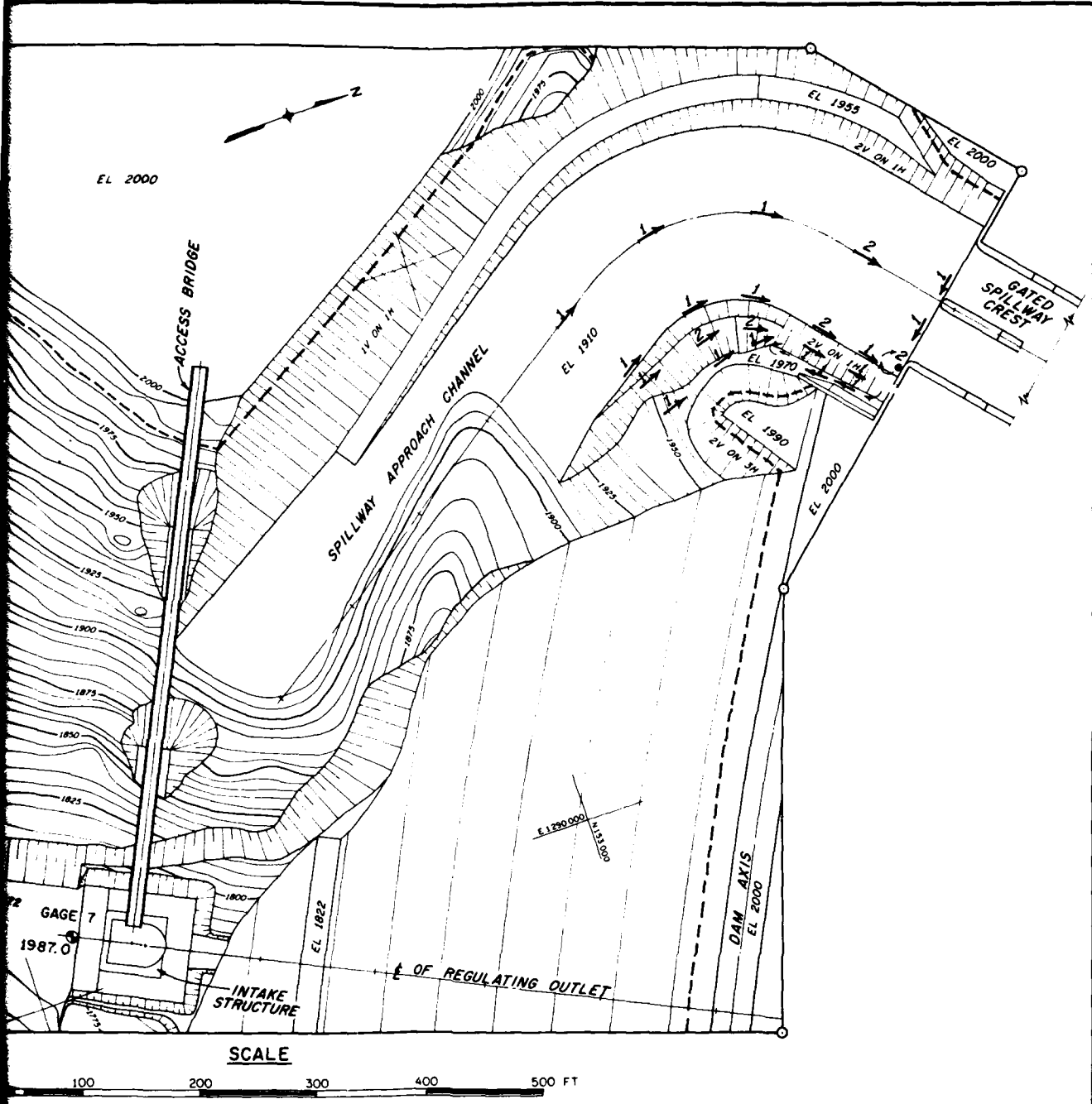


SCALE



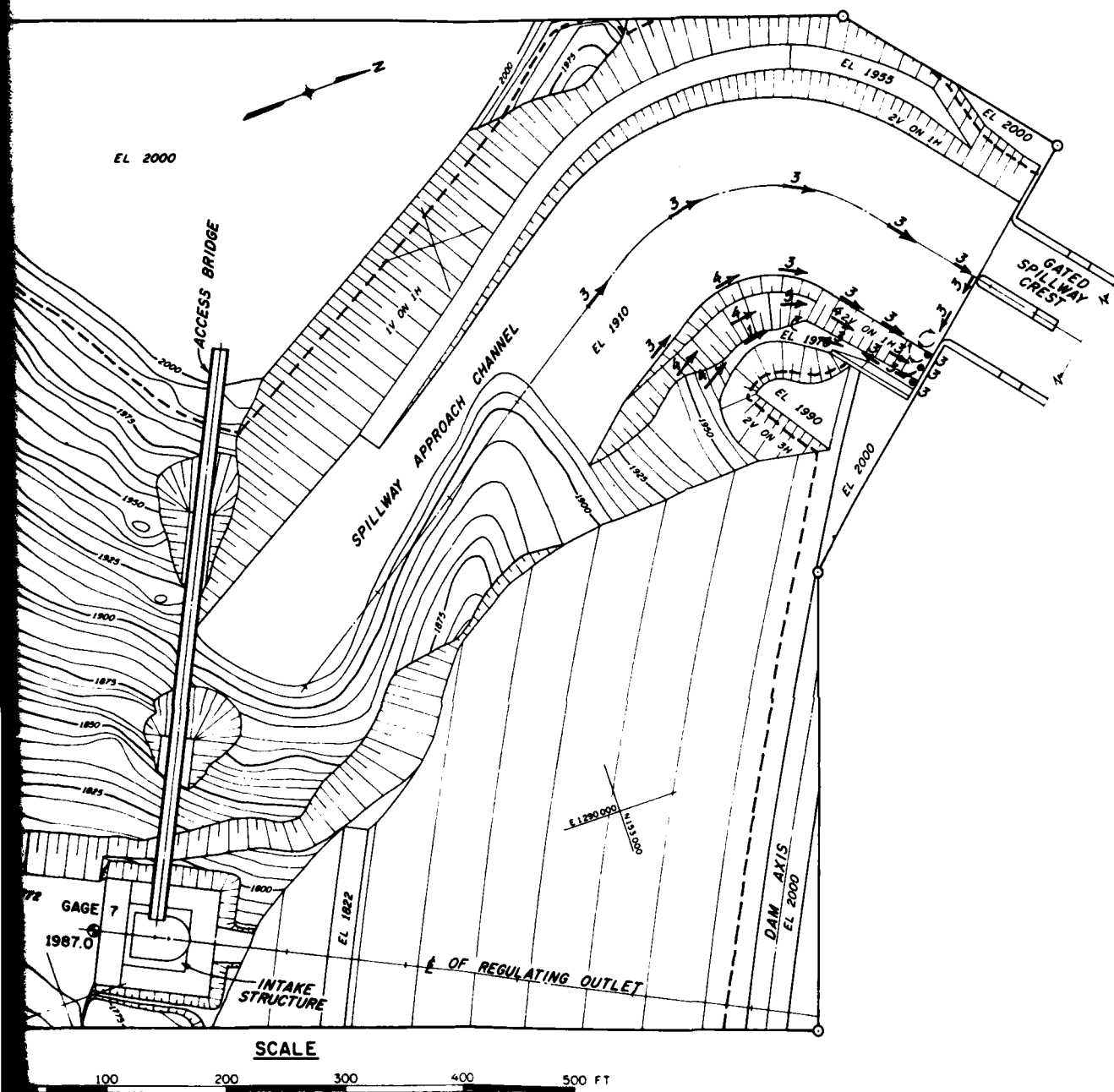
# LEGEND

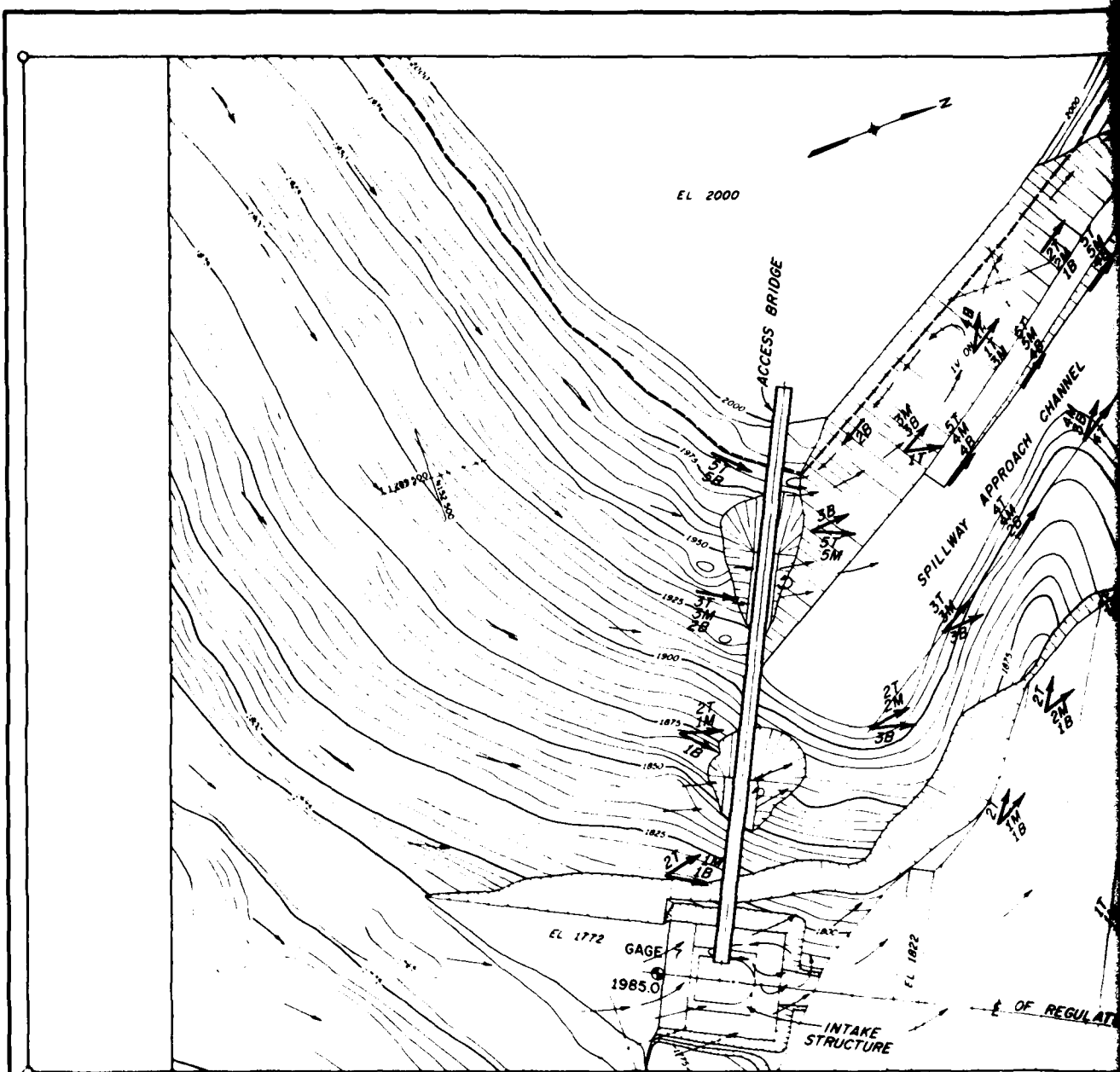
- 3 VELOCITY IN FPS, 3 FT ABOVE BOTTOM
- UPWARD FLOW



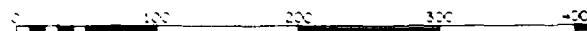
FLOW CONDITIONS  
 FOREBAY  
 SPILLWAY DISCHARGE 18 400 CFS







SCALE

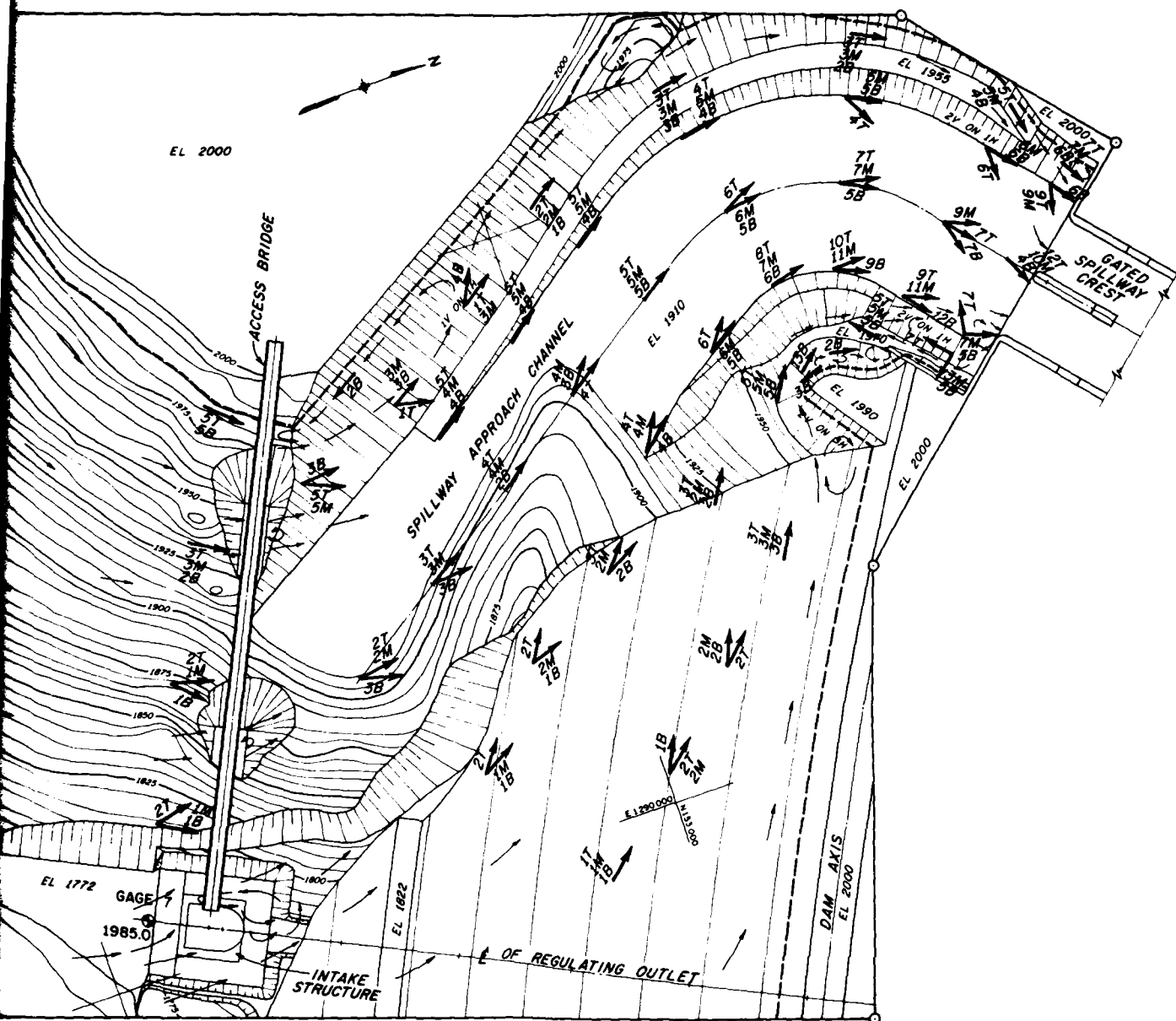


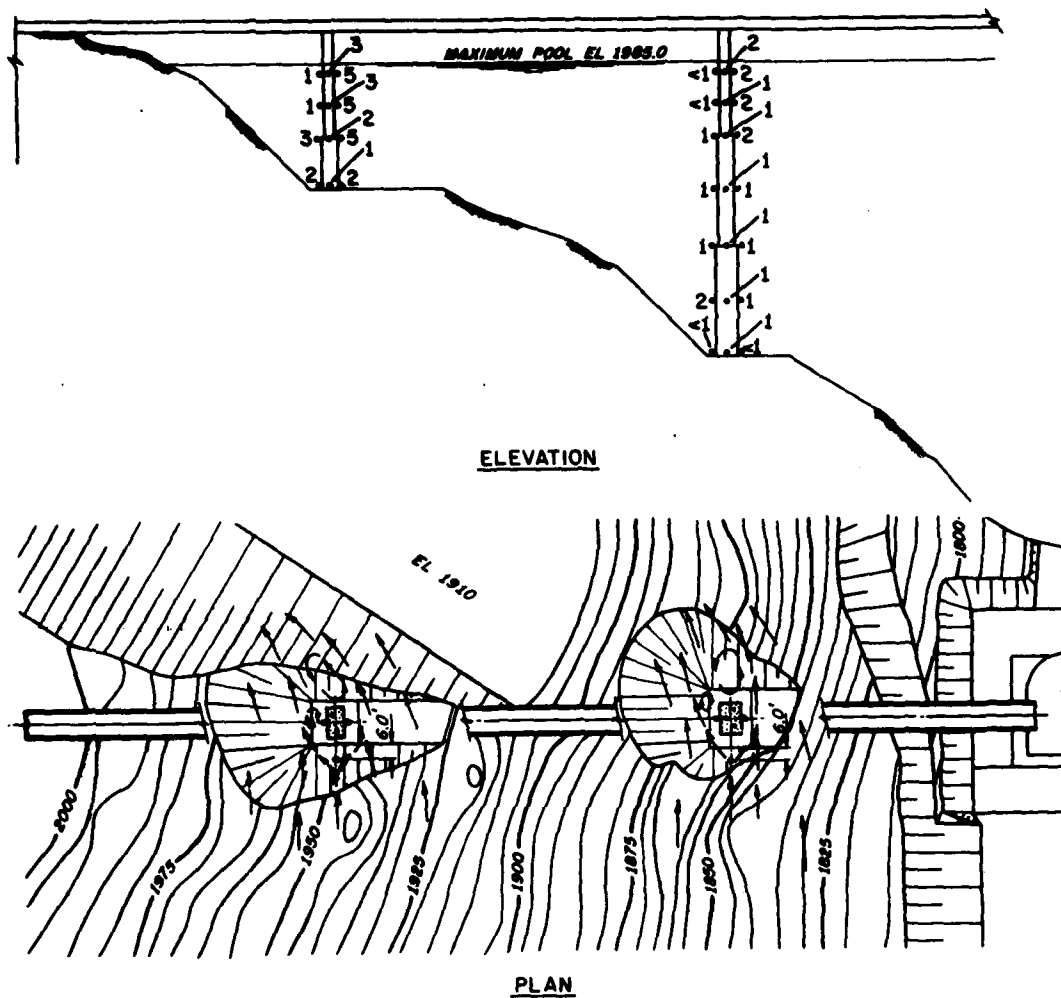
# LEGEND

- VELOCITY IN FPS
- 3 FT DEPTH
- M MID-DEPTH
- B 3 FT ABOVE BOTTOM

# OPERATING CONDITIONS

REGULATING OUTLET	5900 CFS
SPILLWAY	93600 CFS





**LEGEND**

2• VELOCITY IN FPS  
 — FLOW DIRECTION

**FLOW CONDITIONS  
 AT BRIDGE PIERS**

RIVER DISCHARGE 99 500 CFS

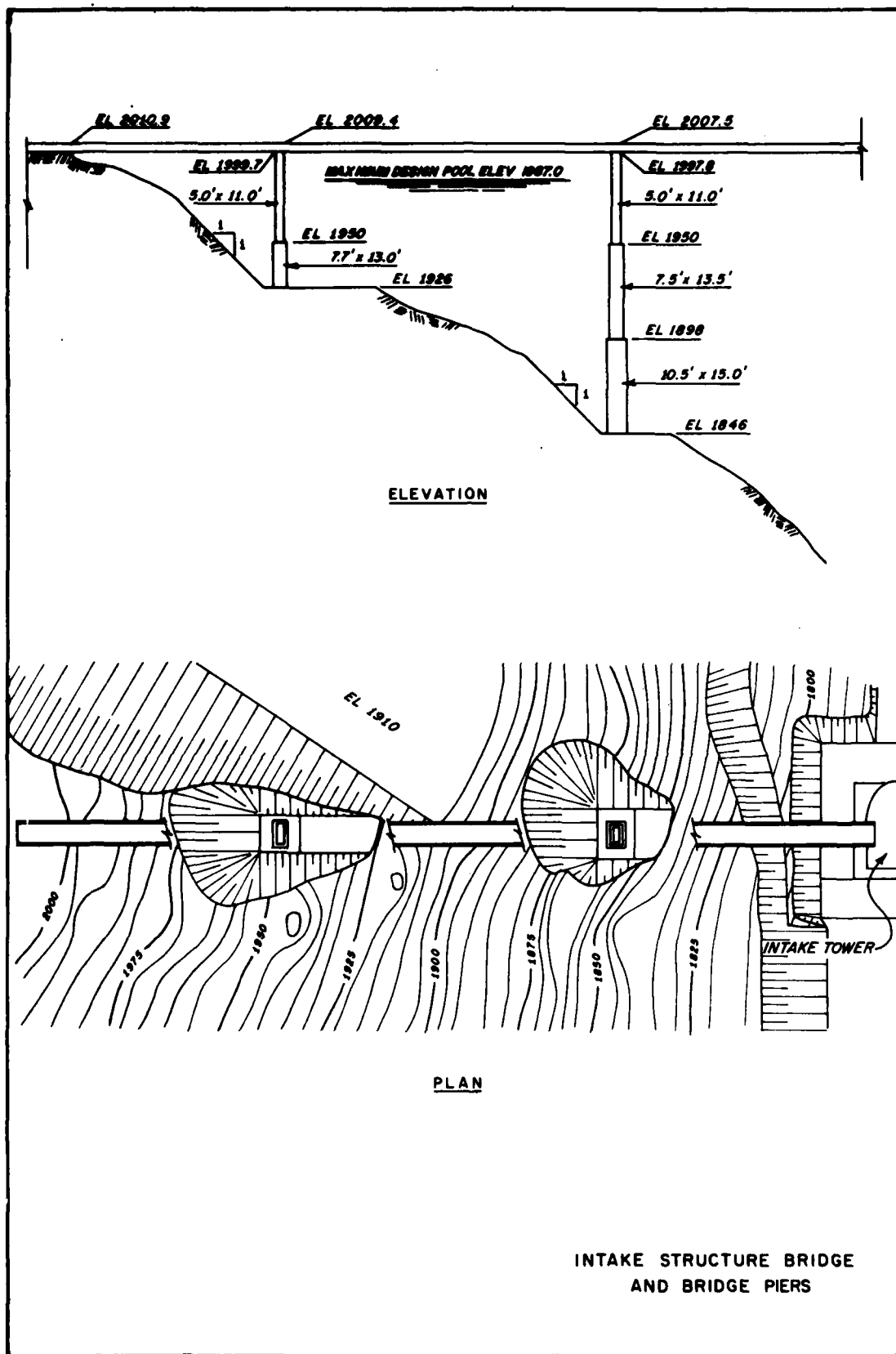
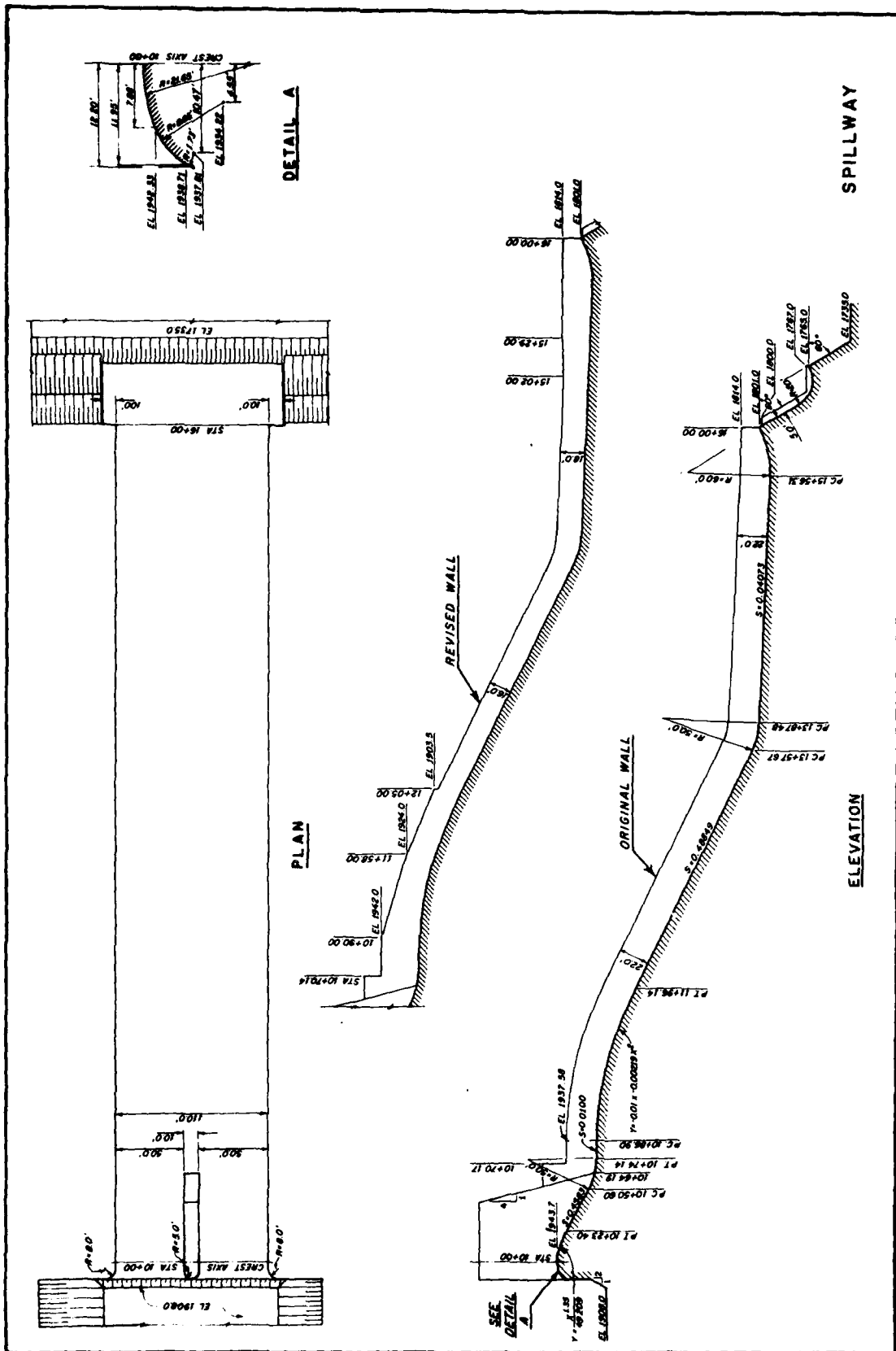
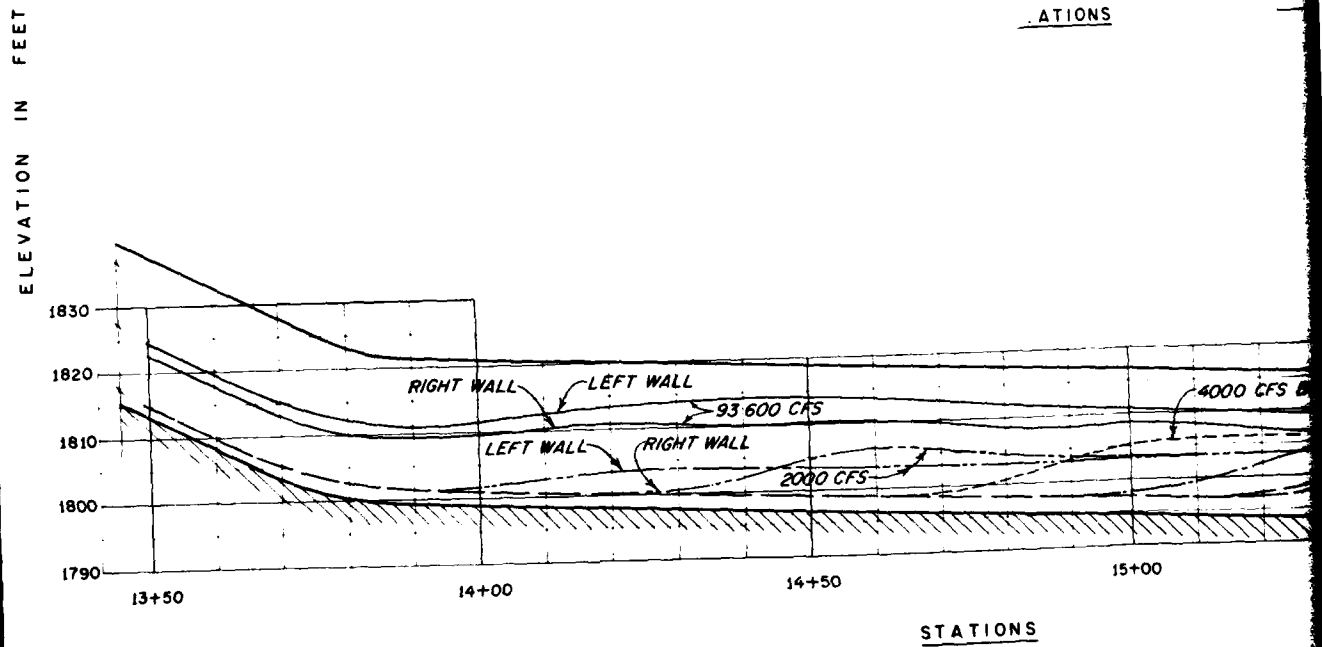
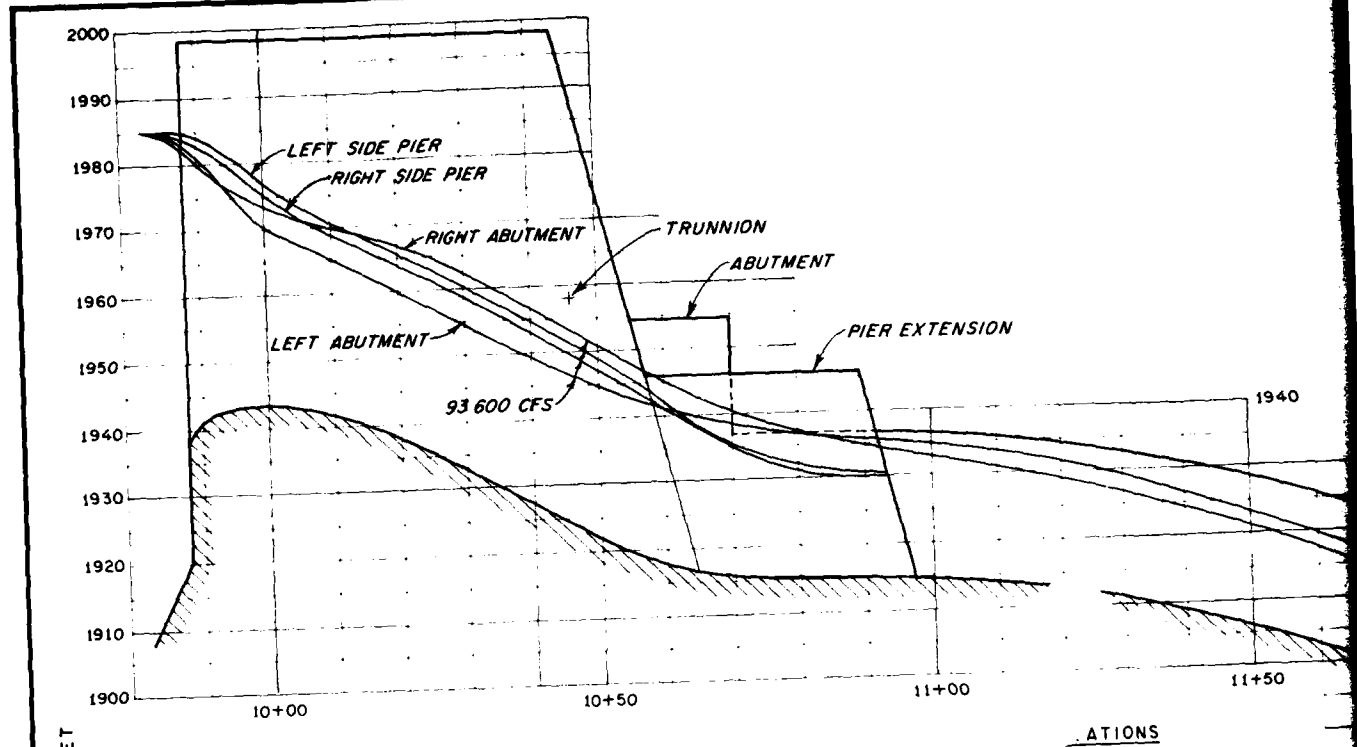
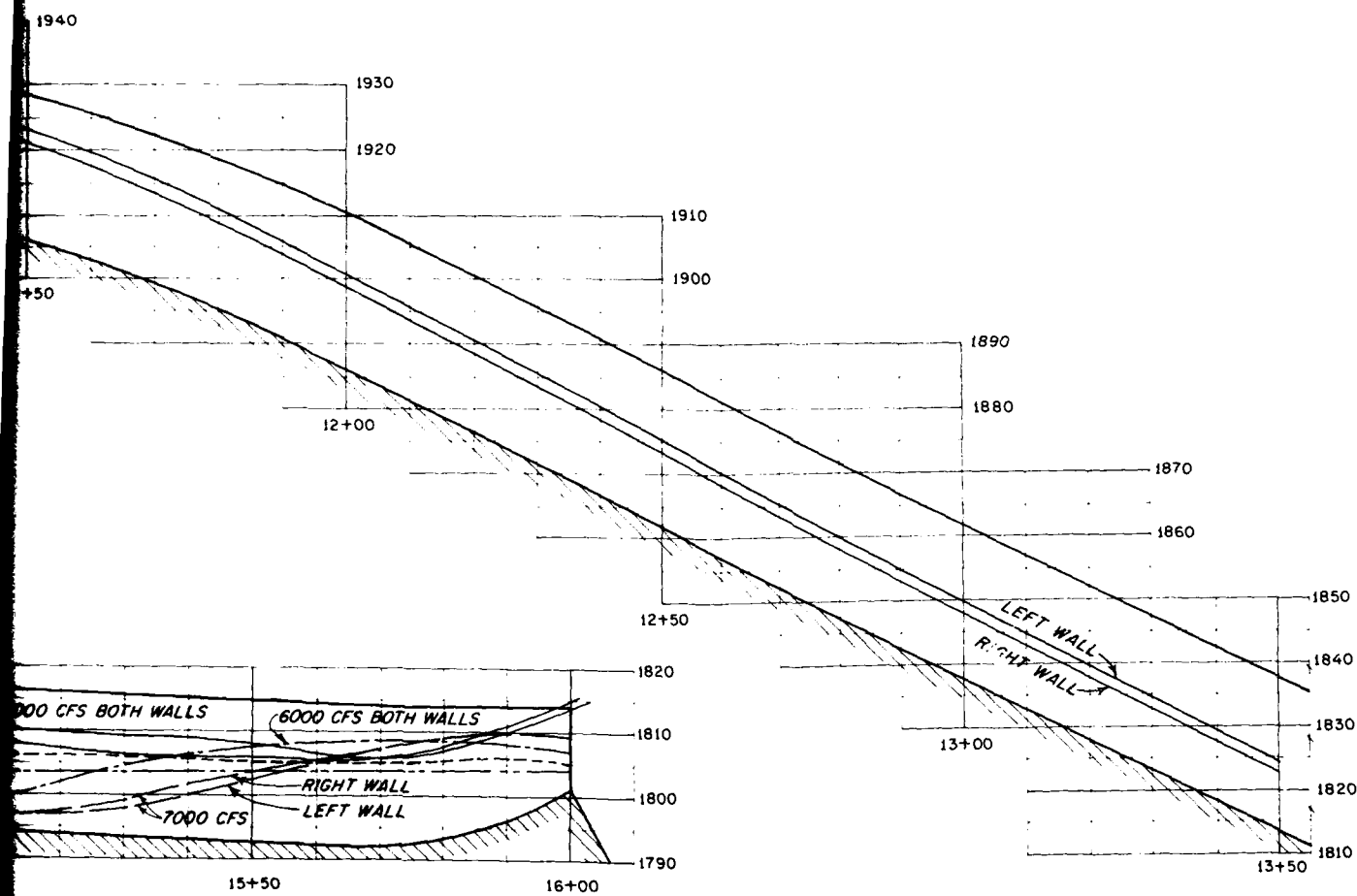


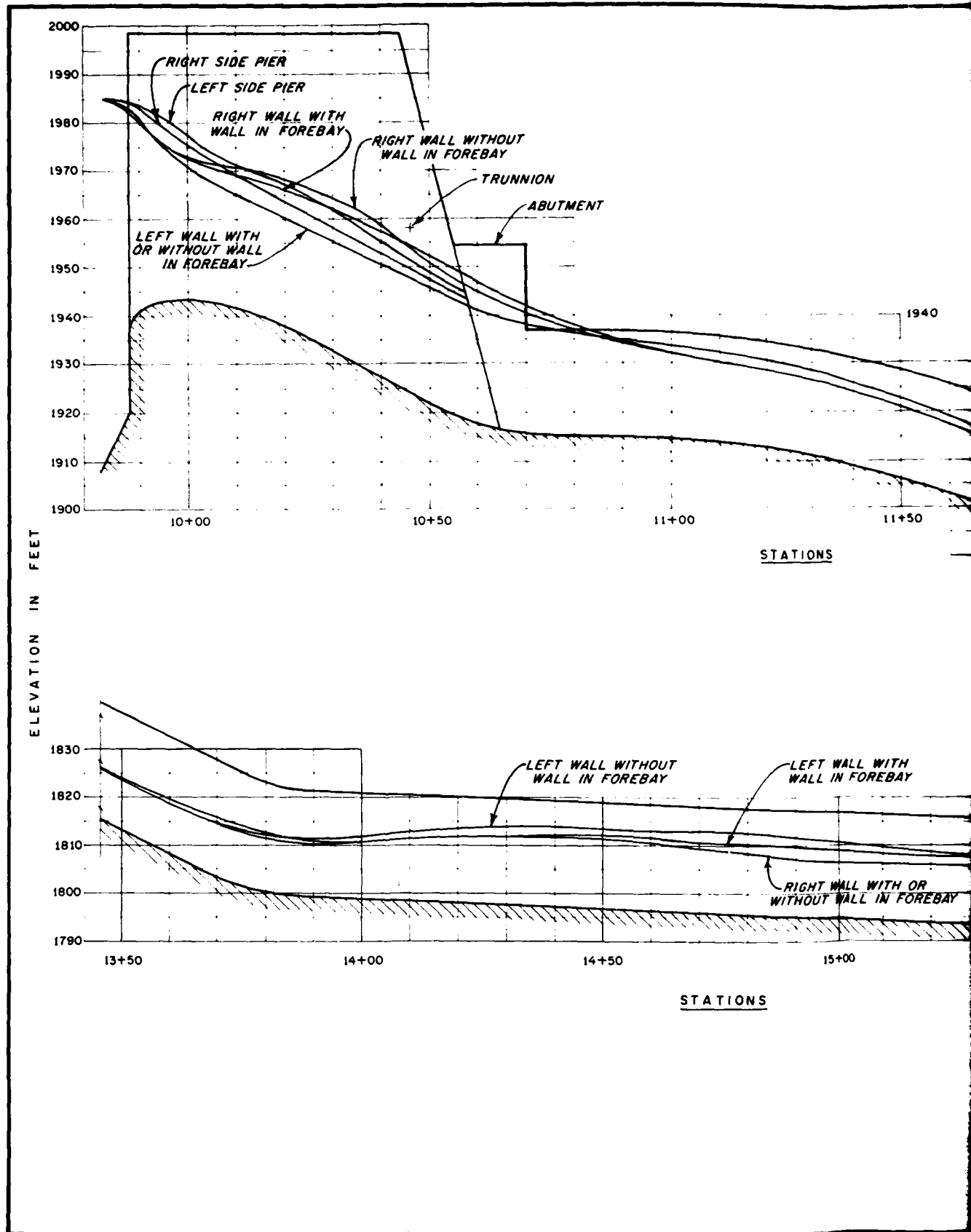
PLATE 7

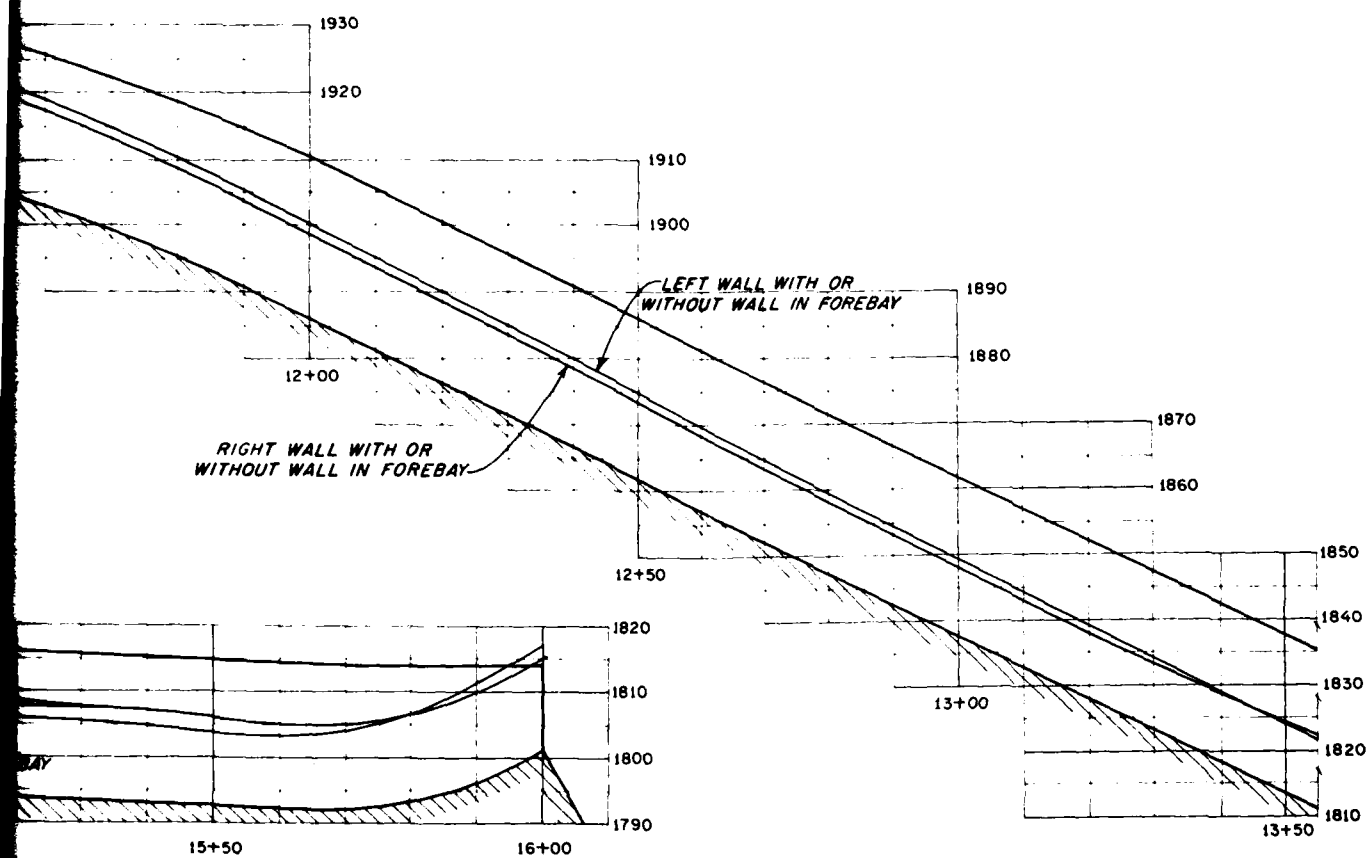




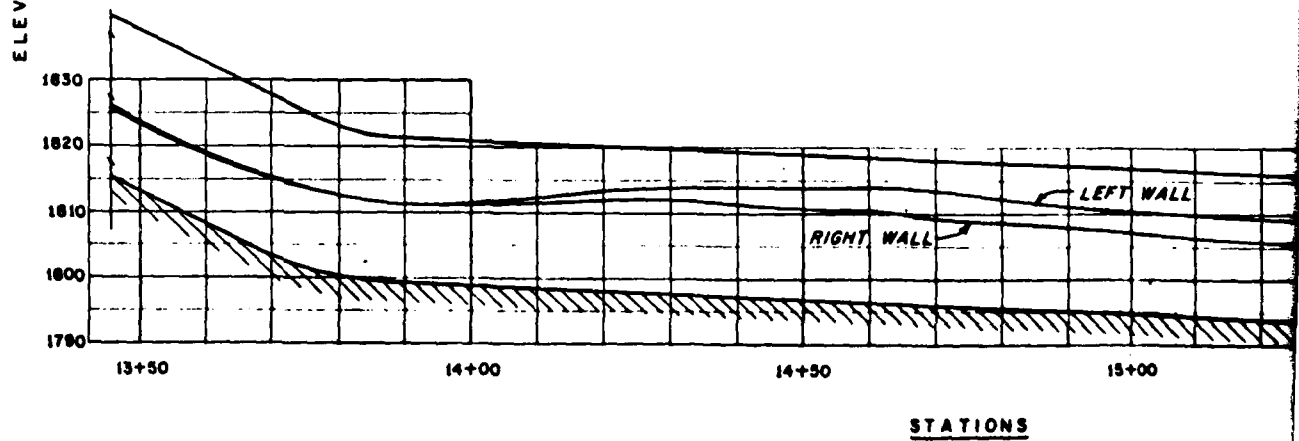
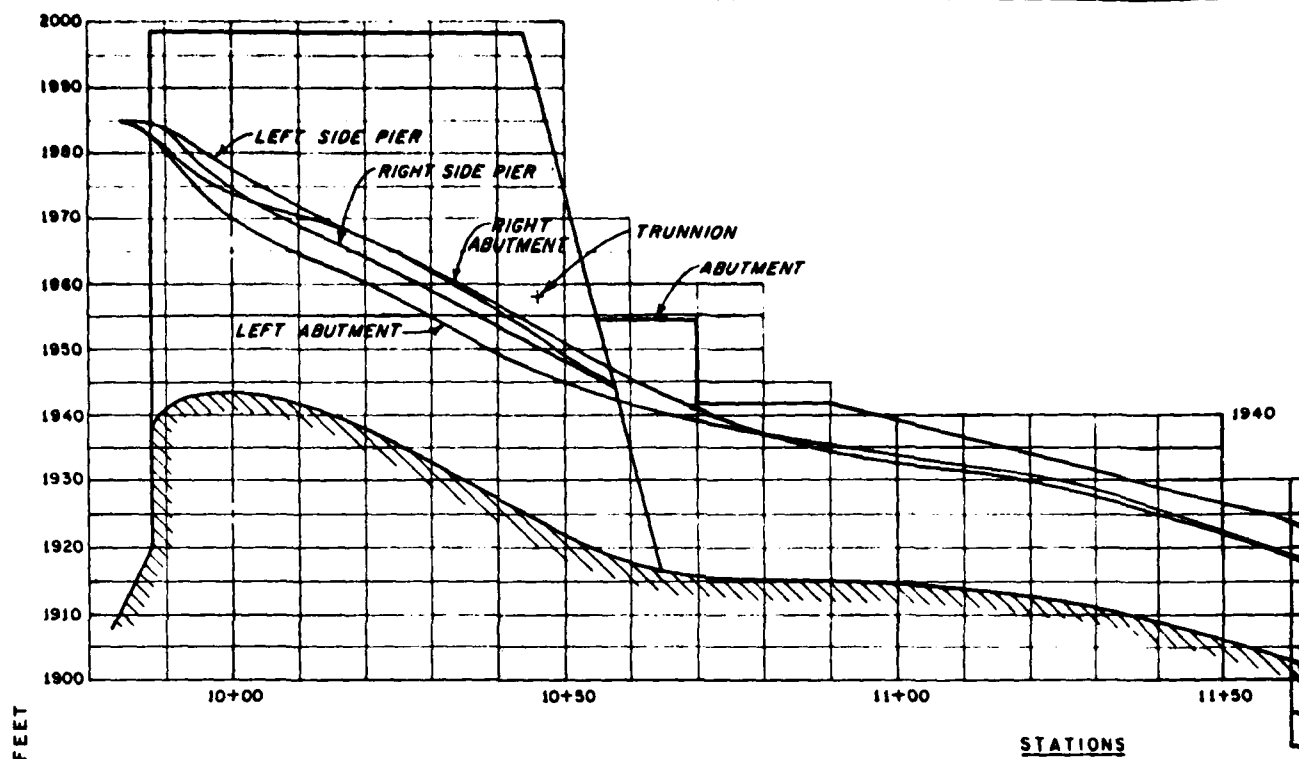


WATER-SURFACE PROFILES  
ON WALLS  
WITH PIER EXTENSION

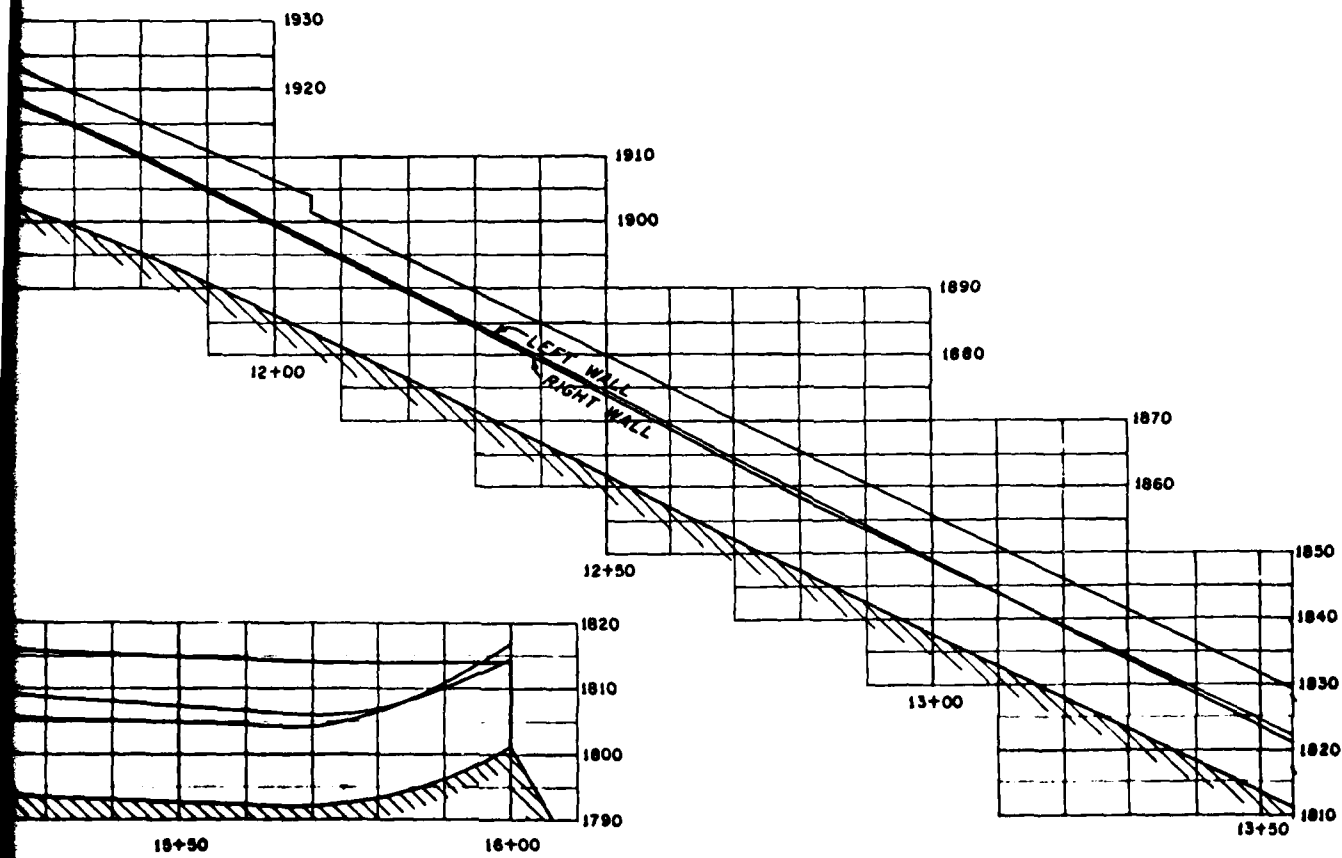




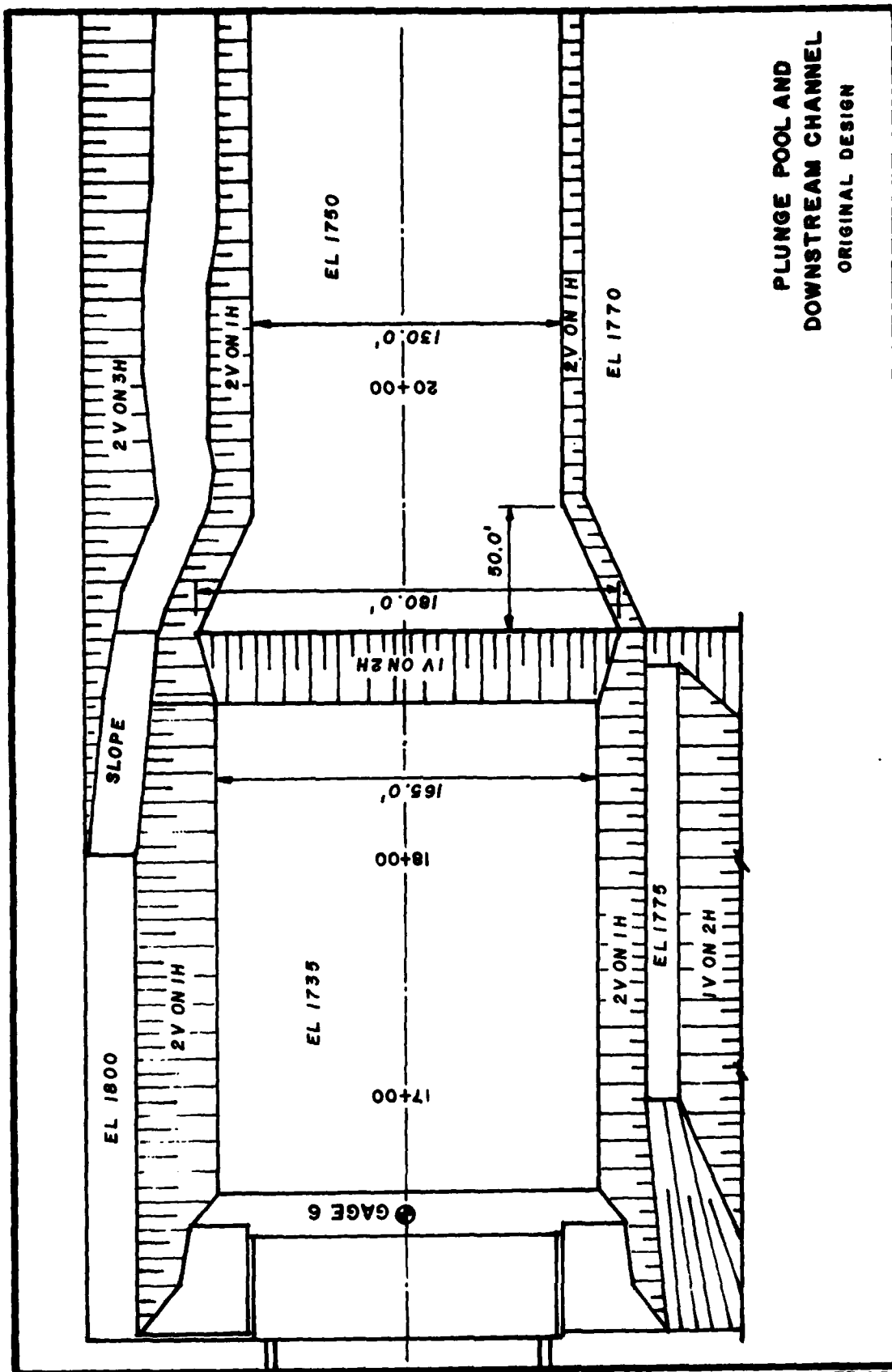
WATER-SURFACE PROFILES  
ON WALLS  
WITH AND WITHOUT FOREBAY WALL  
SPILLWAY DISCHARGE 93600 CFS



2



WATER-SURFACE PROFILES  
ON WALLS  
FINAL DESIGN  
RIVER DISCHARGE 93 600 CFS



PLUNGE POOL AND  
DOWNSTREAM CHANNEL  
ORIGINAL DESIGN



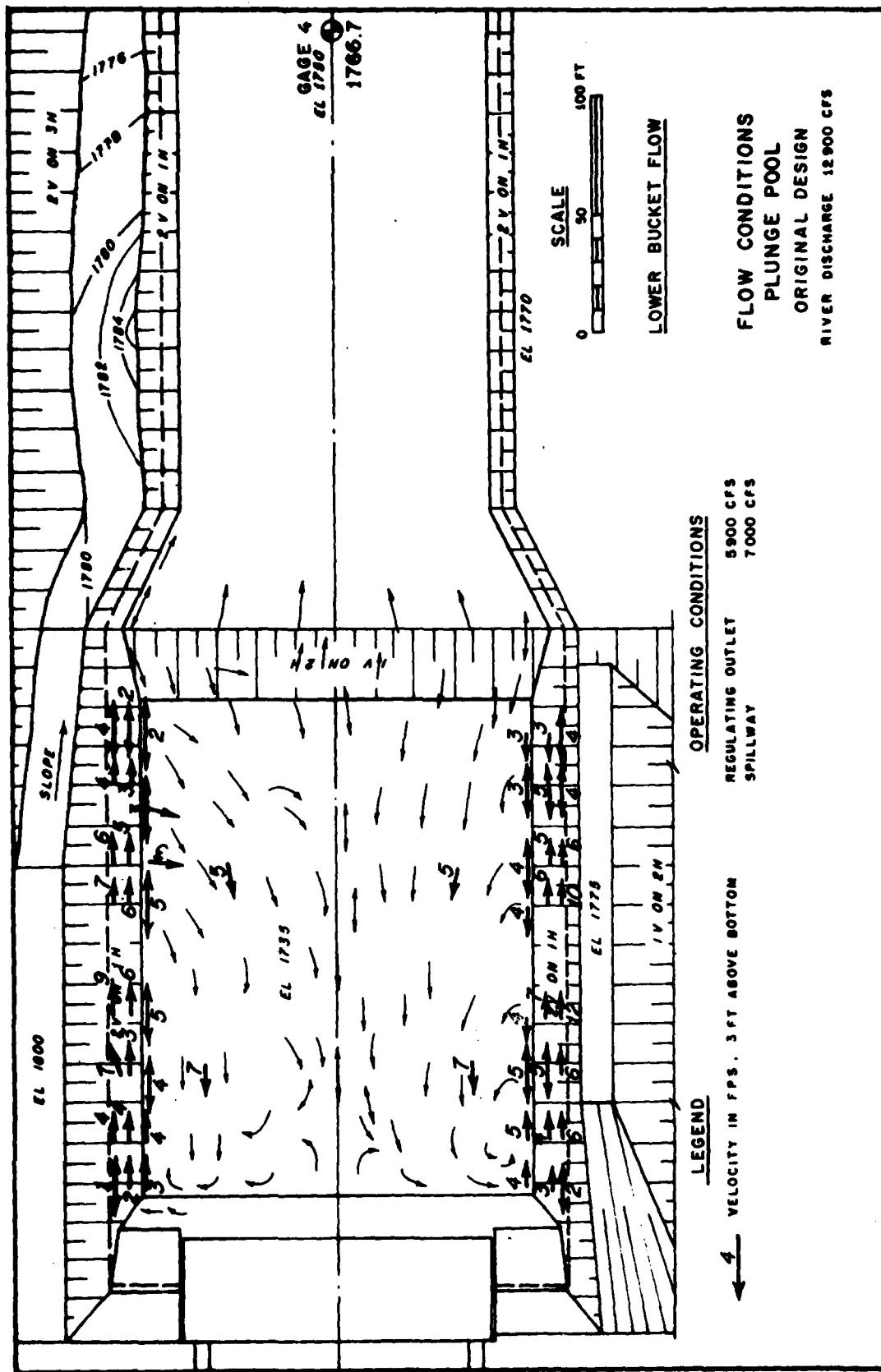


PLATE 15

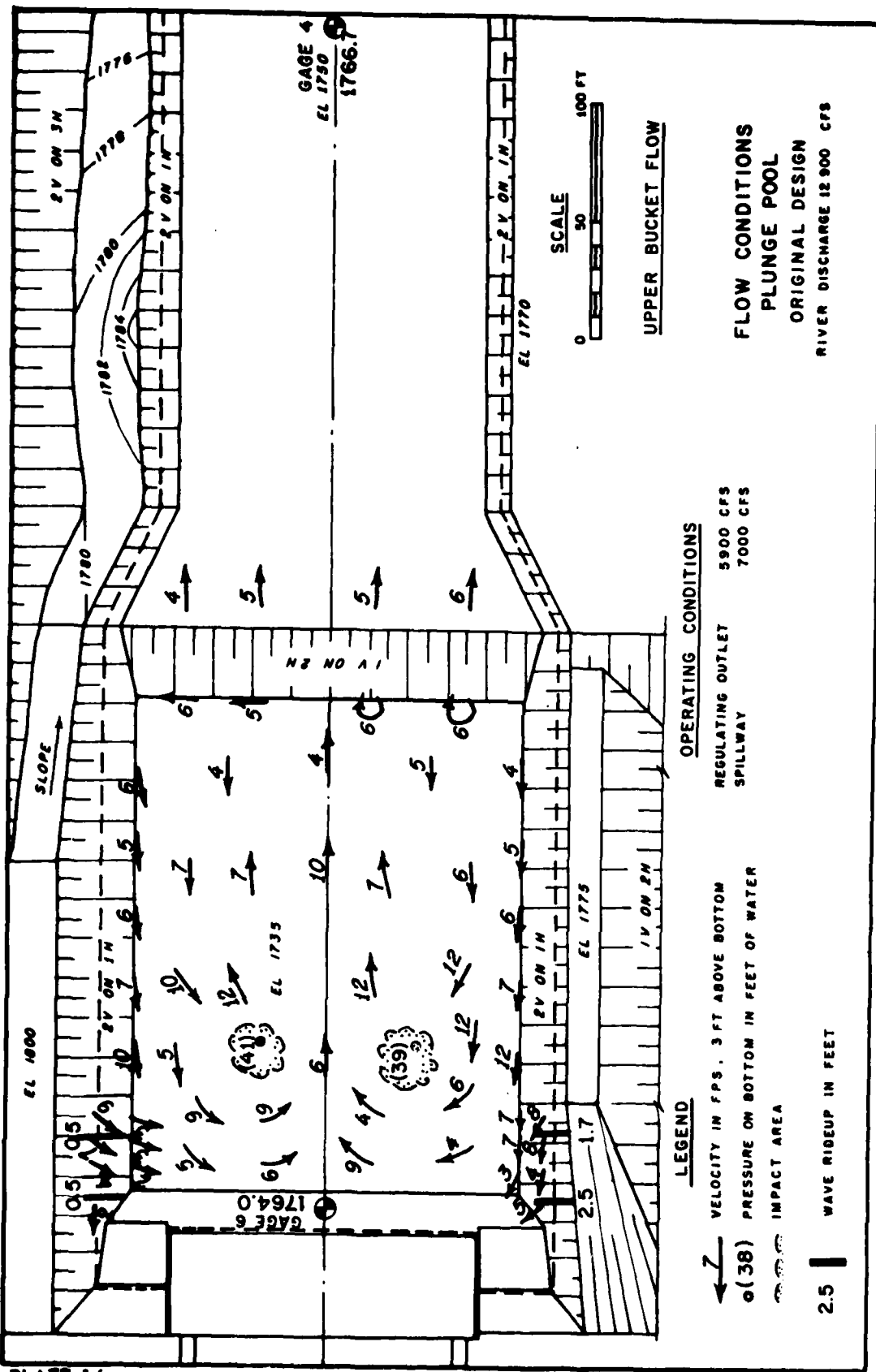
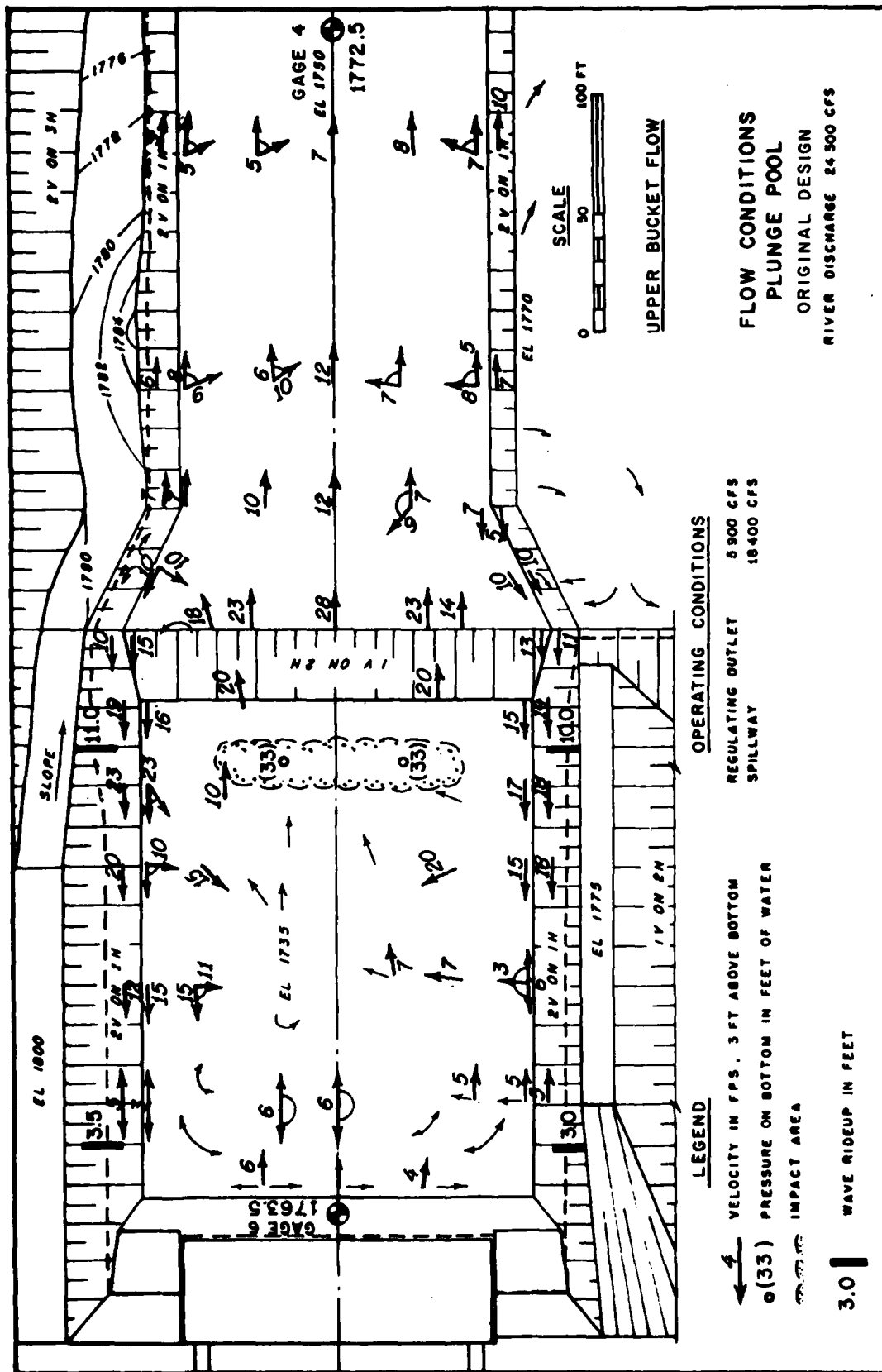
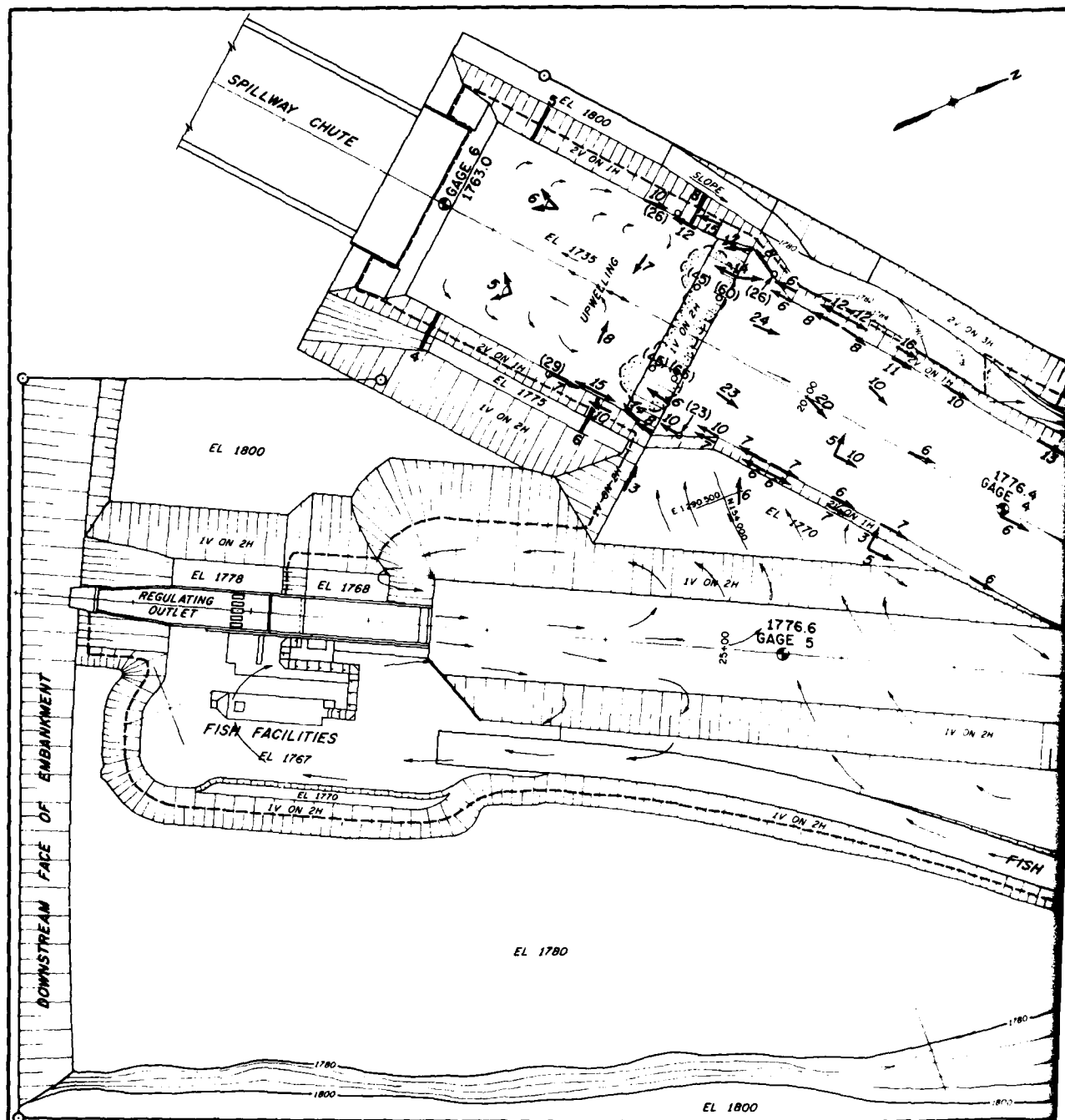


PLATE 16

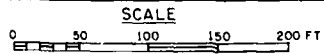
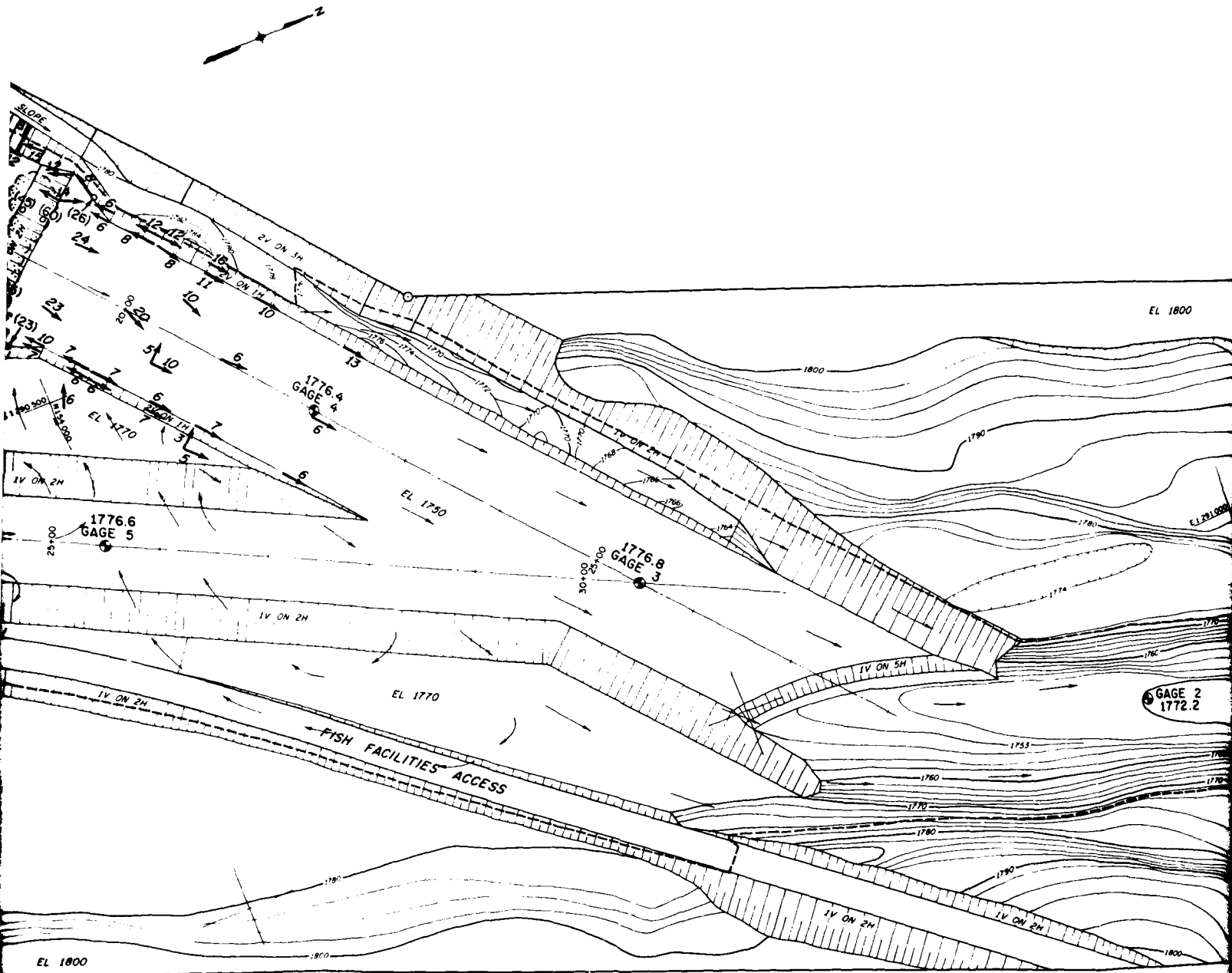




# LEGEND

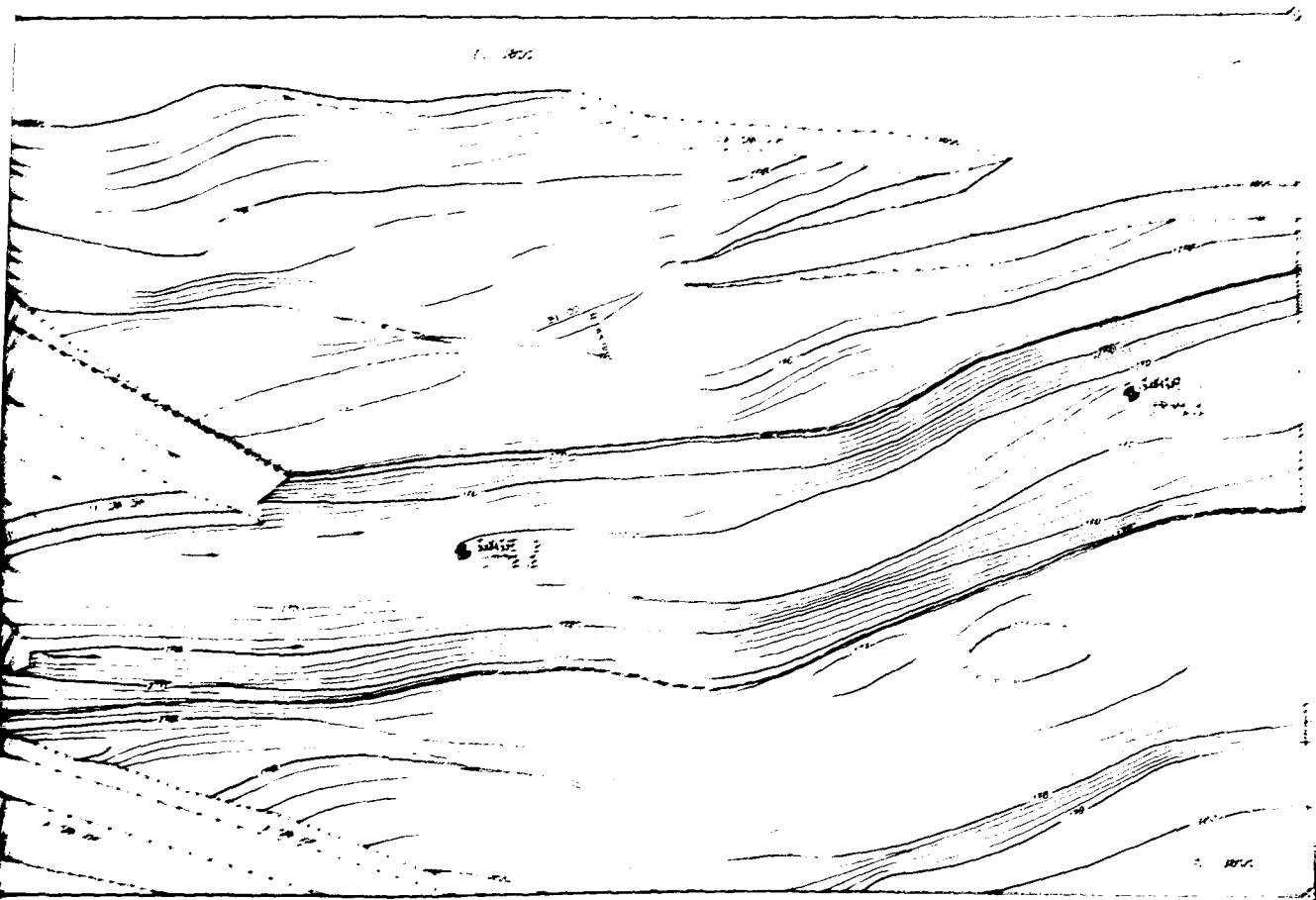
- 13 VELOCITY IN FPS, 3 FT ABOVE BOTTOM
- 0(26) PRESSURE ON BOTTOM IN FT OF WATER
- IMPACT AREA
- 4 WAVE RIDEUP IN FEET

REG  
SPIL



OPERATING CONDITIONS

REGULATING OUTLET	5 900 CFS
SPILLWAY	29 700 CFS



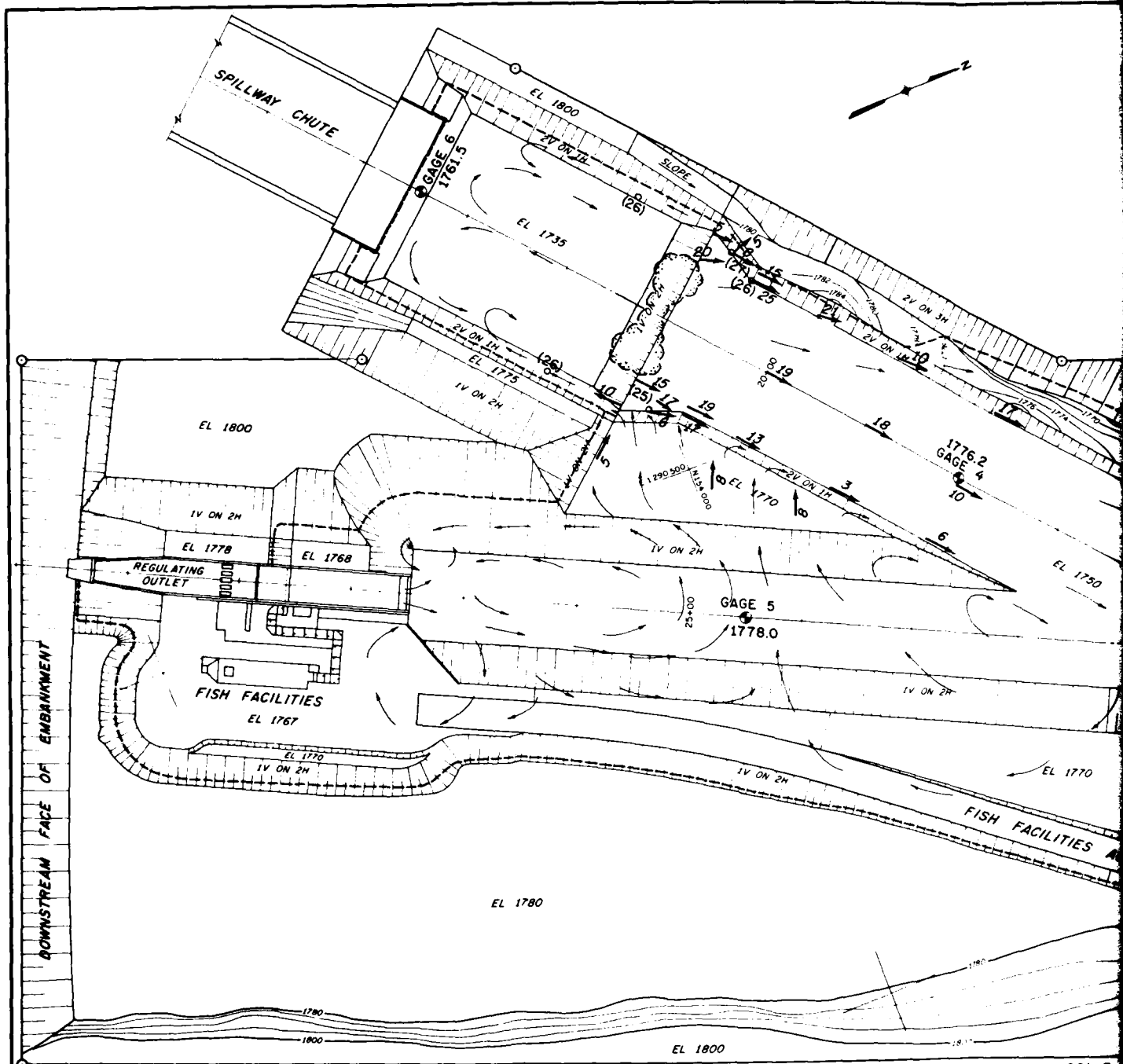
ORIGINAL DESIGN

3.000 3.000 3.000

SANDSTONE

SHALE

PLATE 3

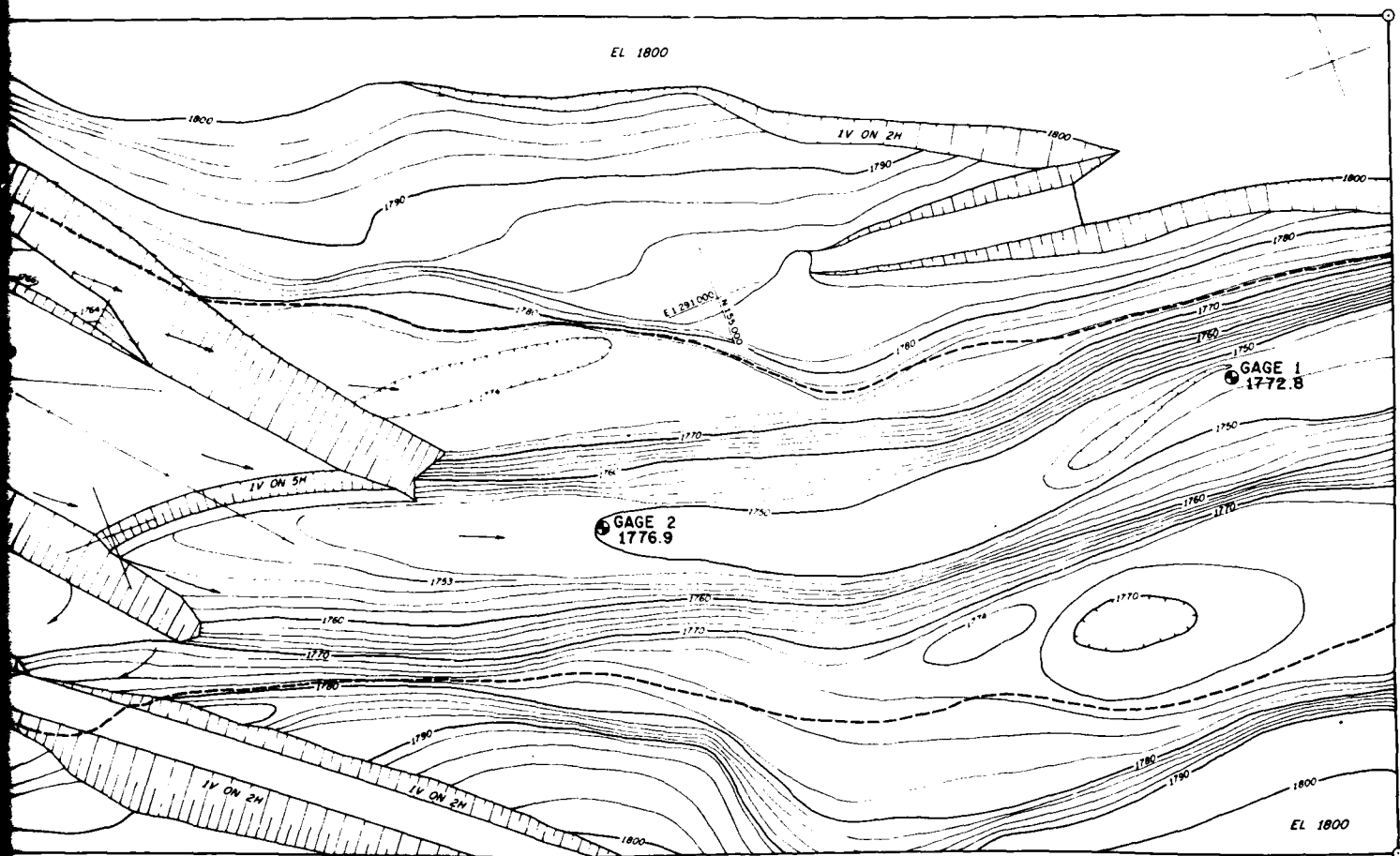


# **LEGEND**

- 13** VELOCITY IN FPS, 3 FT ABOVE BOTTOM
- o(26)** PRESSURE ON BOTTOM IN FT OF WATER
- ☉☉☉** IMPACT AREA

# **OPERATING C**

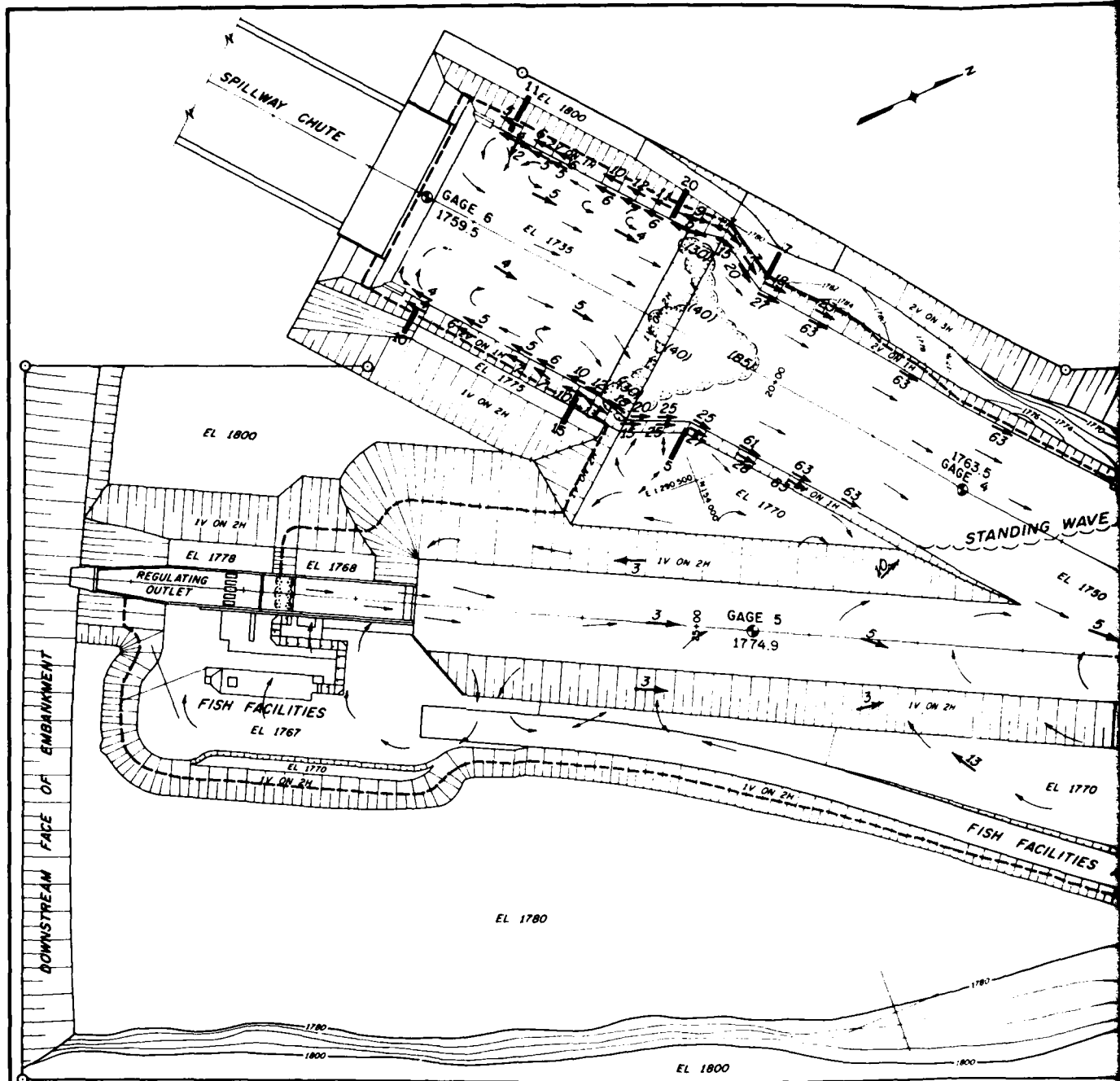
- REGULATING OUTLET
- SPILLWAY




ORIGINAL DESIGN

FLOW CONDITIONS  
SPILLWAY CHANNEL  
RIVER DISCHARGE 49800 CFS

PLATE 19



# **LEGEND**

- $\frac{13}{\leftarrow}$  VELOCITY IN FPS, 3 FT ABOVE BOTTOM
- (26) PRESSURE ON BOTTOM IN FT OF WATER
-  IMPACT AREA
- $\frac{5}{\leftarrow}$  WAVE RIDEUP IN FEET

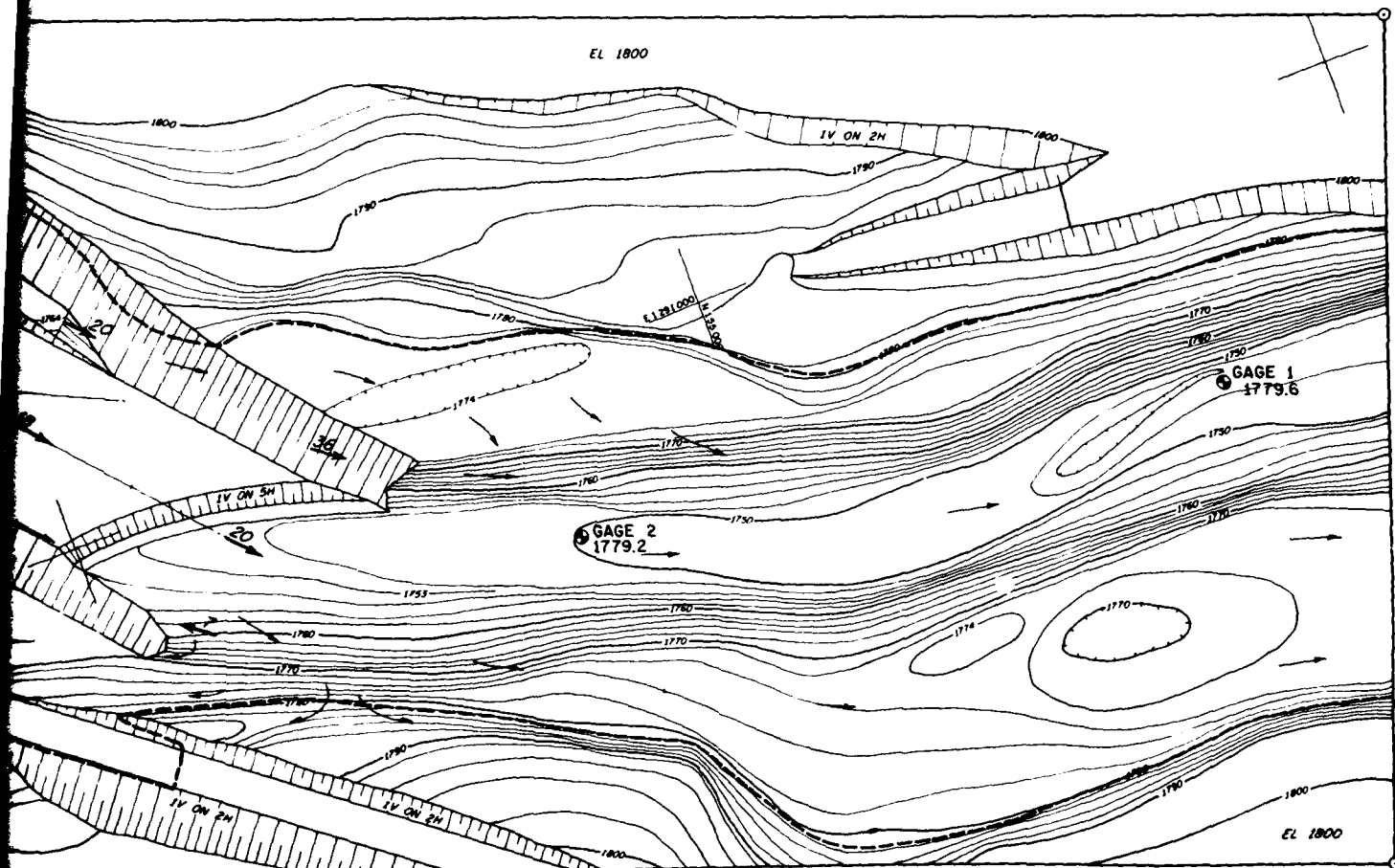
# **OPERATING CO**

REGULATING OUTLET  
SPILLWAY

2

SCALE

0 100 200 300 400 500 FT



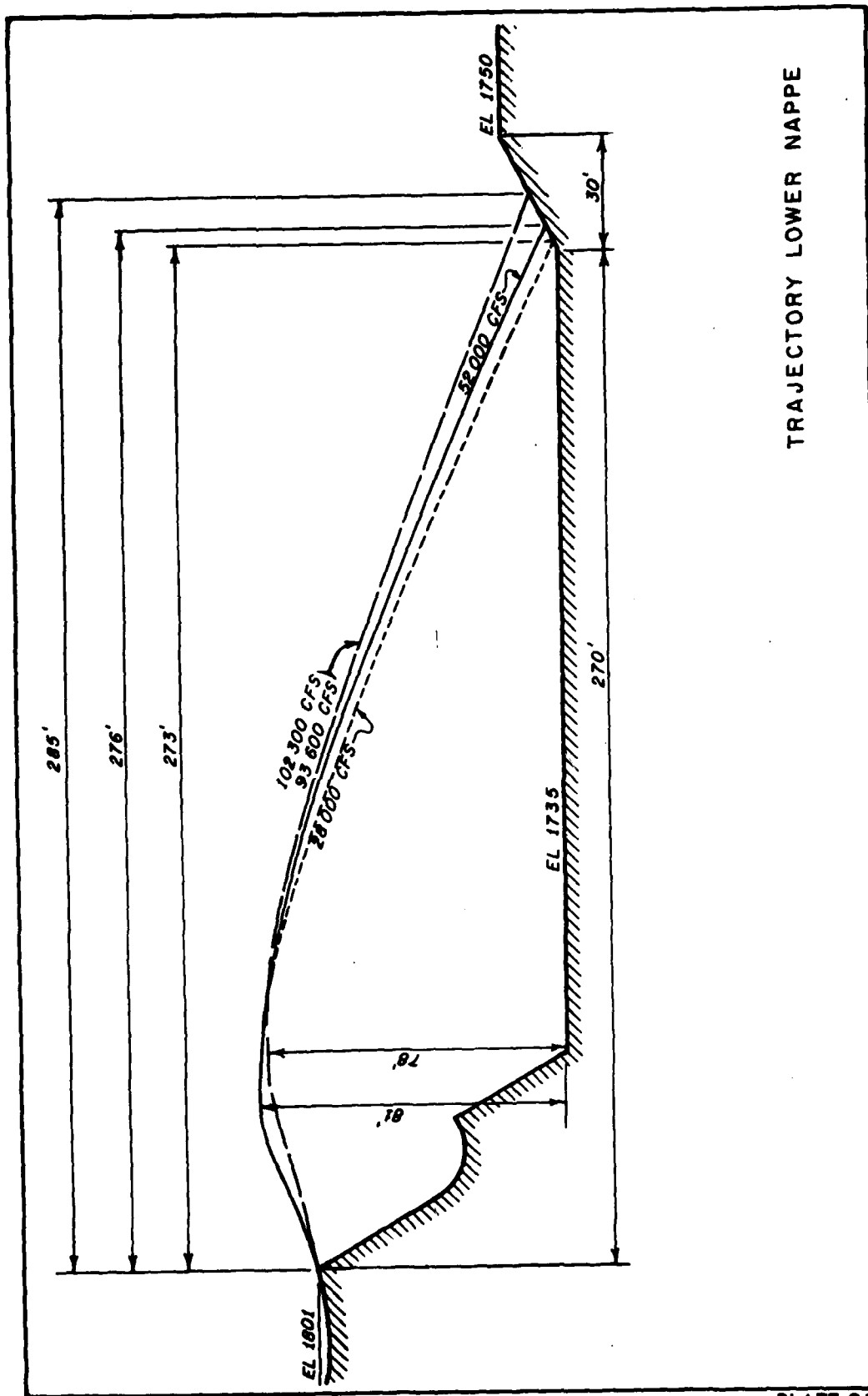
ORIGINAL DESIGN

FLOW CONDITIONS

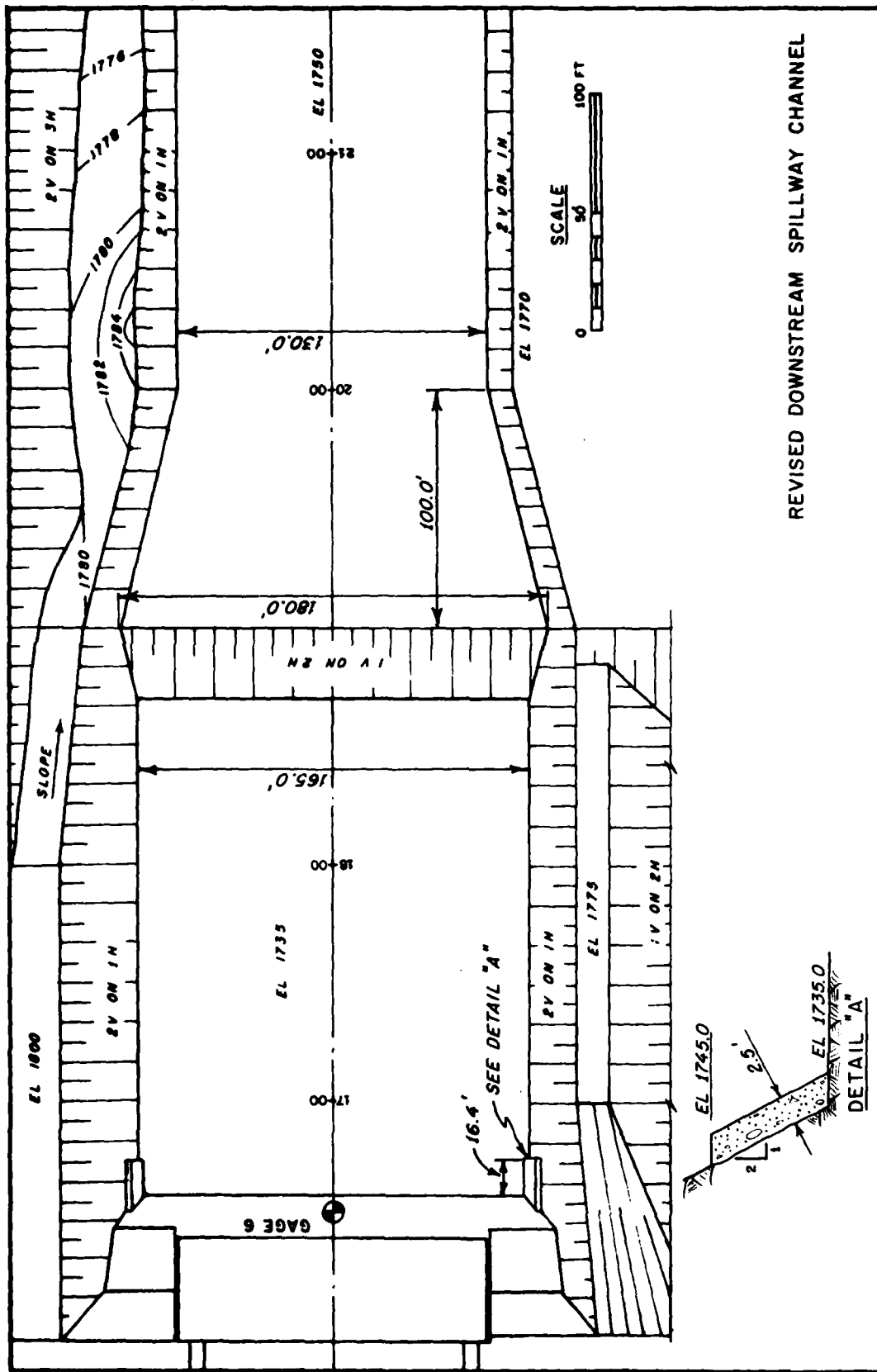
SPILLWAY CHANNEL

RIVER DISCHARGE 99 500 CFS

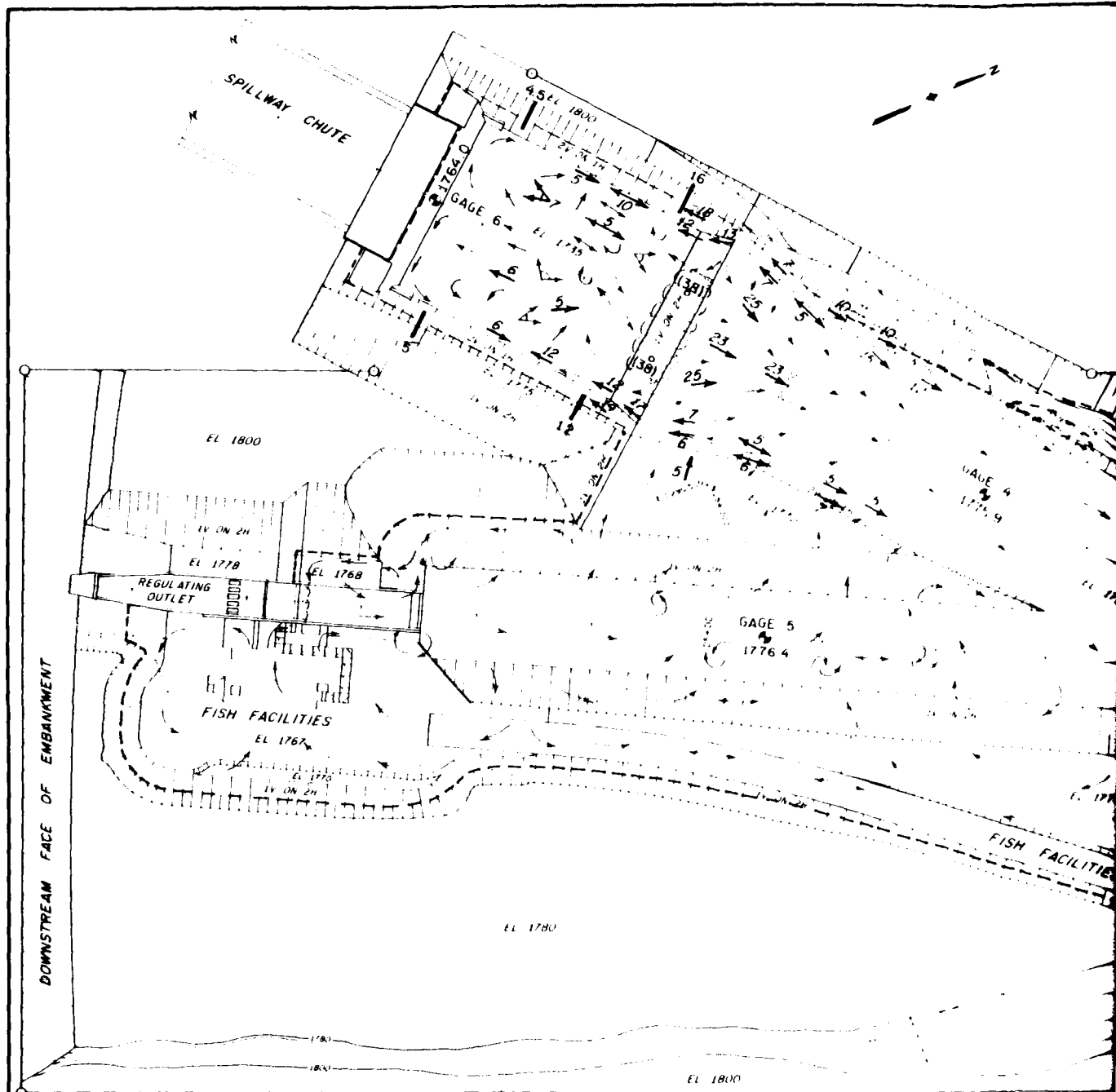
PLATE 20



TRAJECTORY LOWER NAPPE



REVISED DOWNSTREAM SPILLWAY CHANNEL



# LEGEND

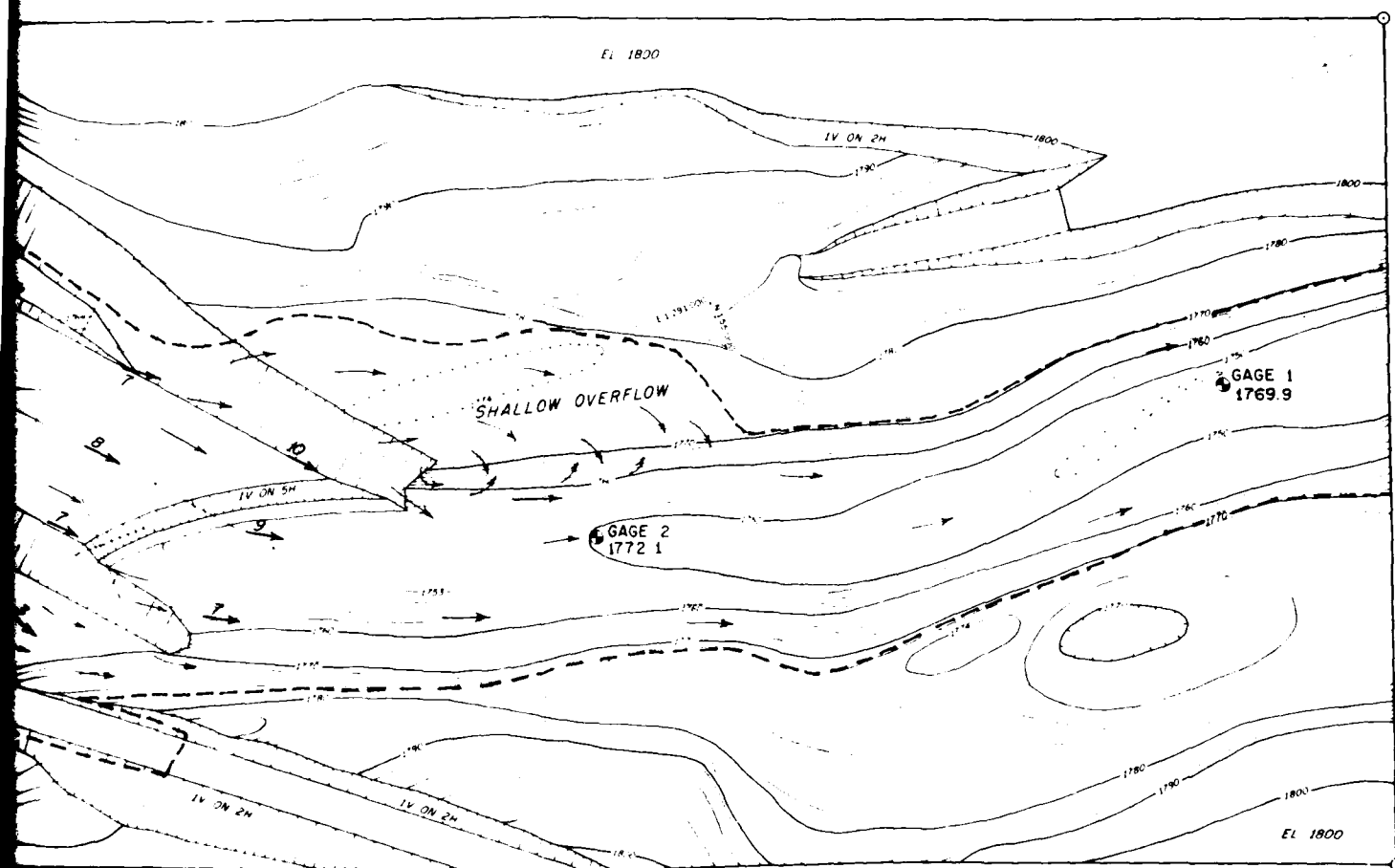
- 13 VELOCITY IN FPS, 3 FT ABOVE BOTTOM
- 26 PRESSURE ON BOTTOM IN FT OF WATER
- 5 IMPACT AREA
- 5 WAVE RIDEUP IN FEET

# OPERATING

- REGULATING OUTLET
- SPILLWAY

2

SCALE



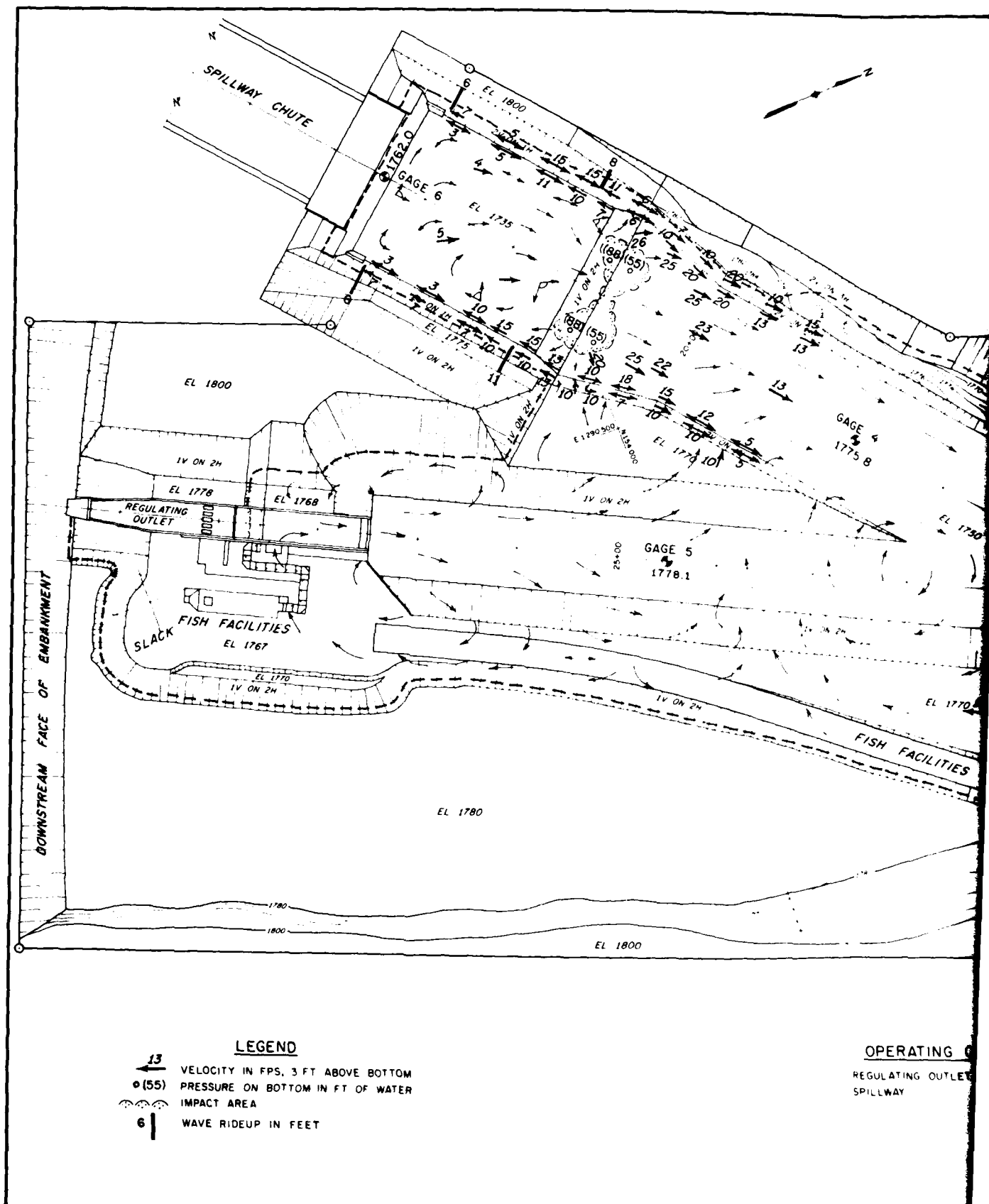
REVISED DOWNSTREAM CHANNEL

FLOW CONDITIONS

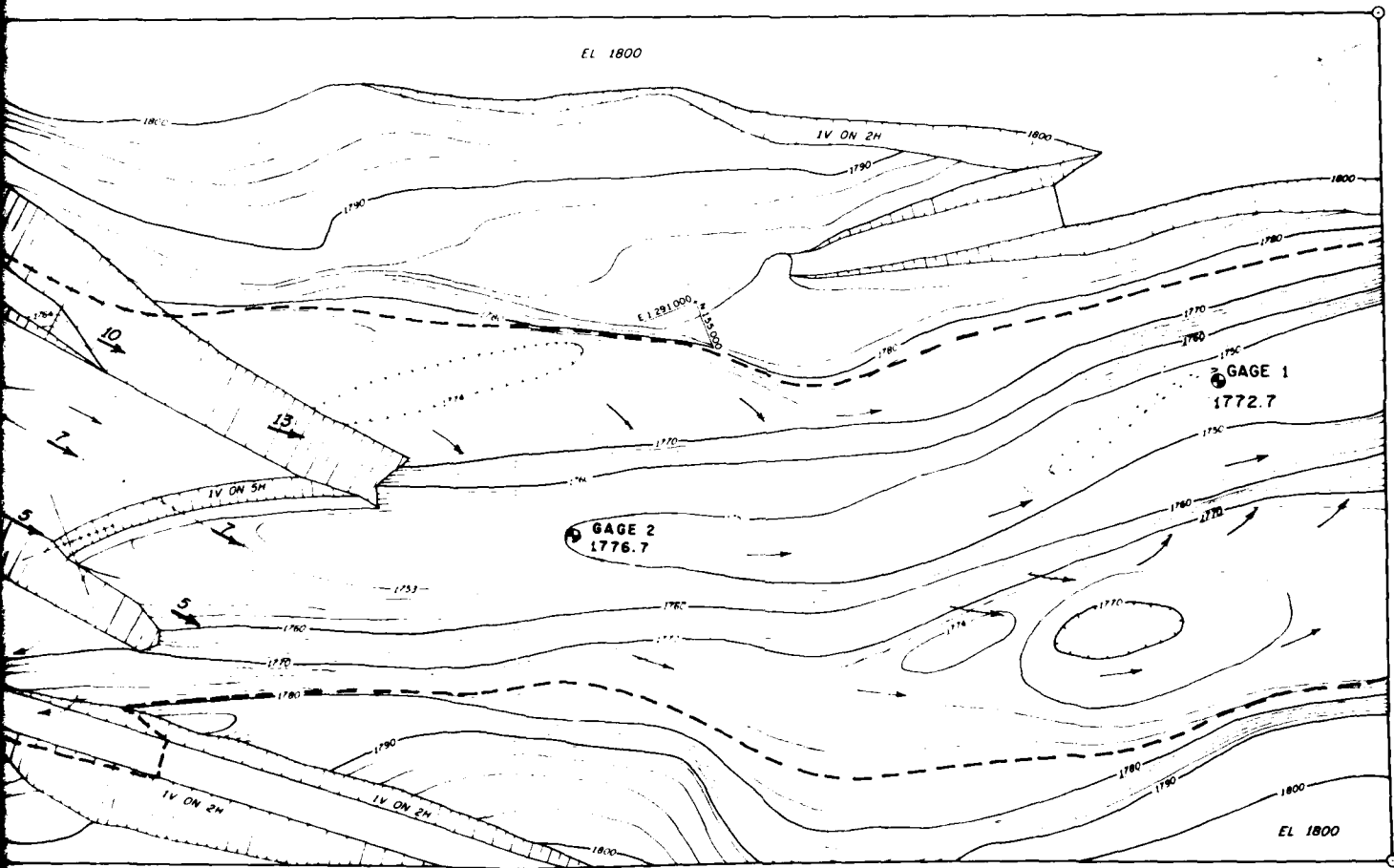
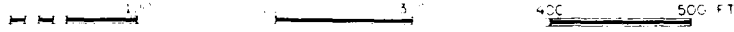
SPILLWAY CHANNEL

RIVER DISCHARGE 35 600 CFS

PLATE 23



SCALE



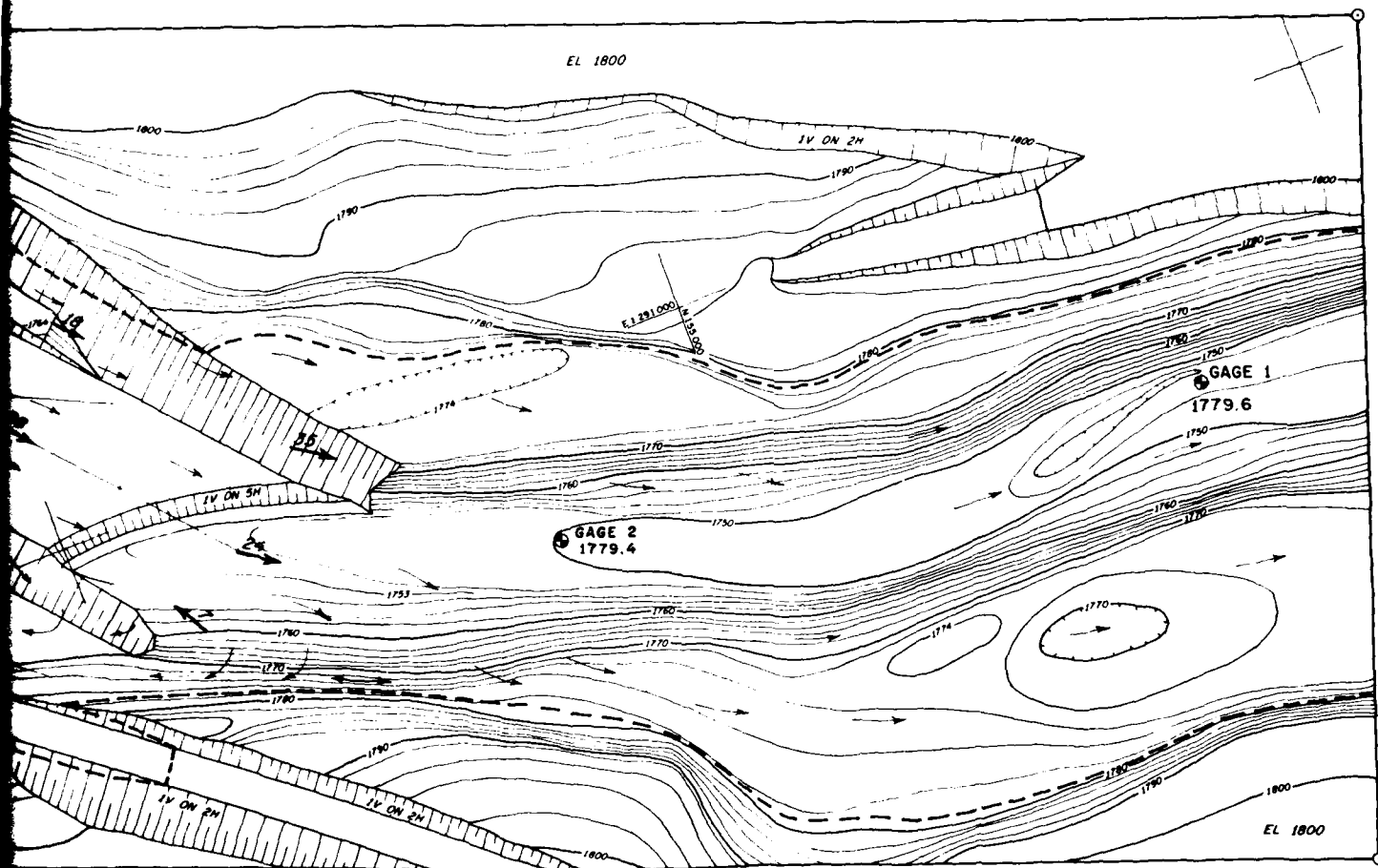
REVISED DOWNSTREAM CHANNEL

FLOW CONDITIONS  
 SPILLWAY CHANNEL  
 RIVER DISCHARGE 49 800 CFS



2

SCALE

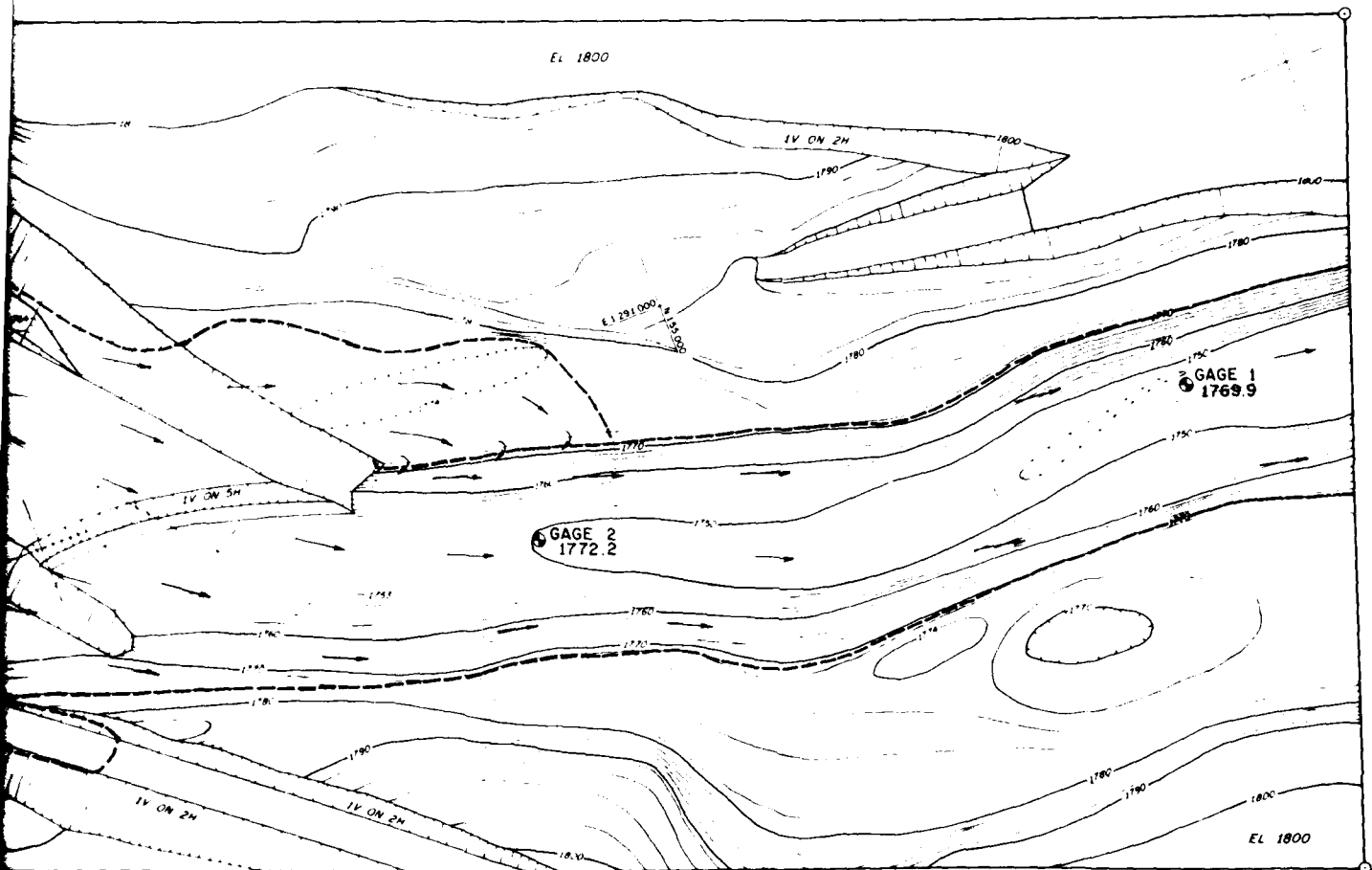


REVISED DOWNSTREAM CHANNEL

FLOW CONDITIONS  
SPILLWAY CHANNEL  
RIVER DISCHARGE 99 500 CFS



SCALE

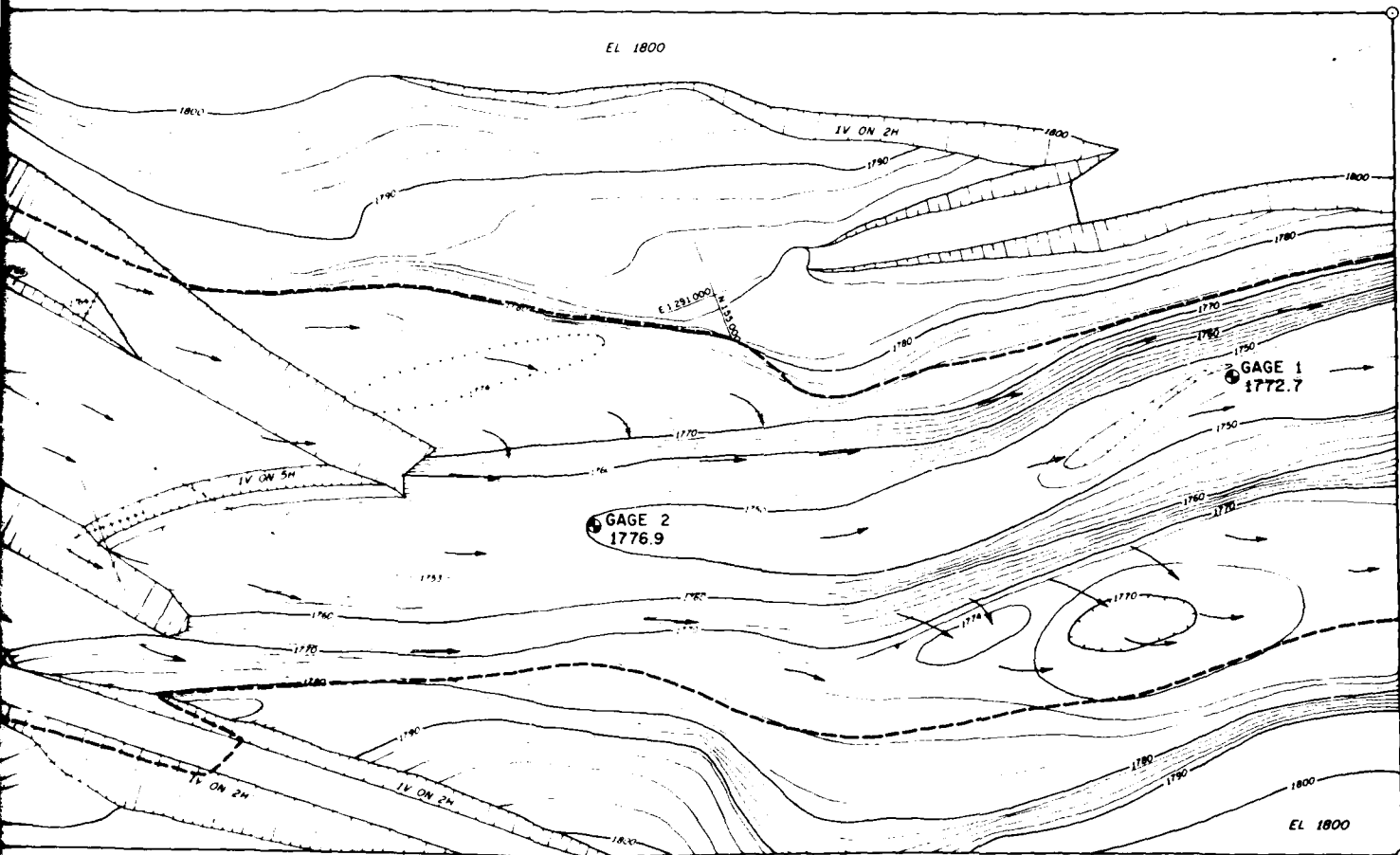
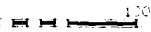


PLAN B PLUNGE POOL

FLOW CONDITIONS  
 SPILLWAY CHANNEL  
 RIVER DISCHARGE 35 600 CFS



SCALE



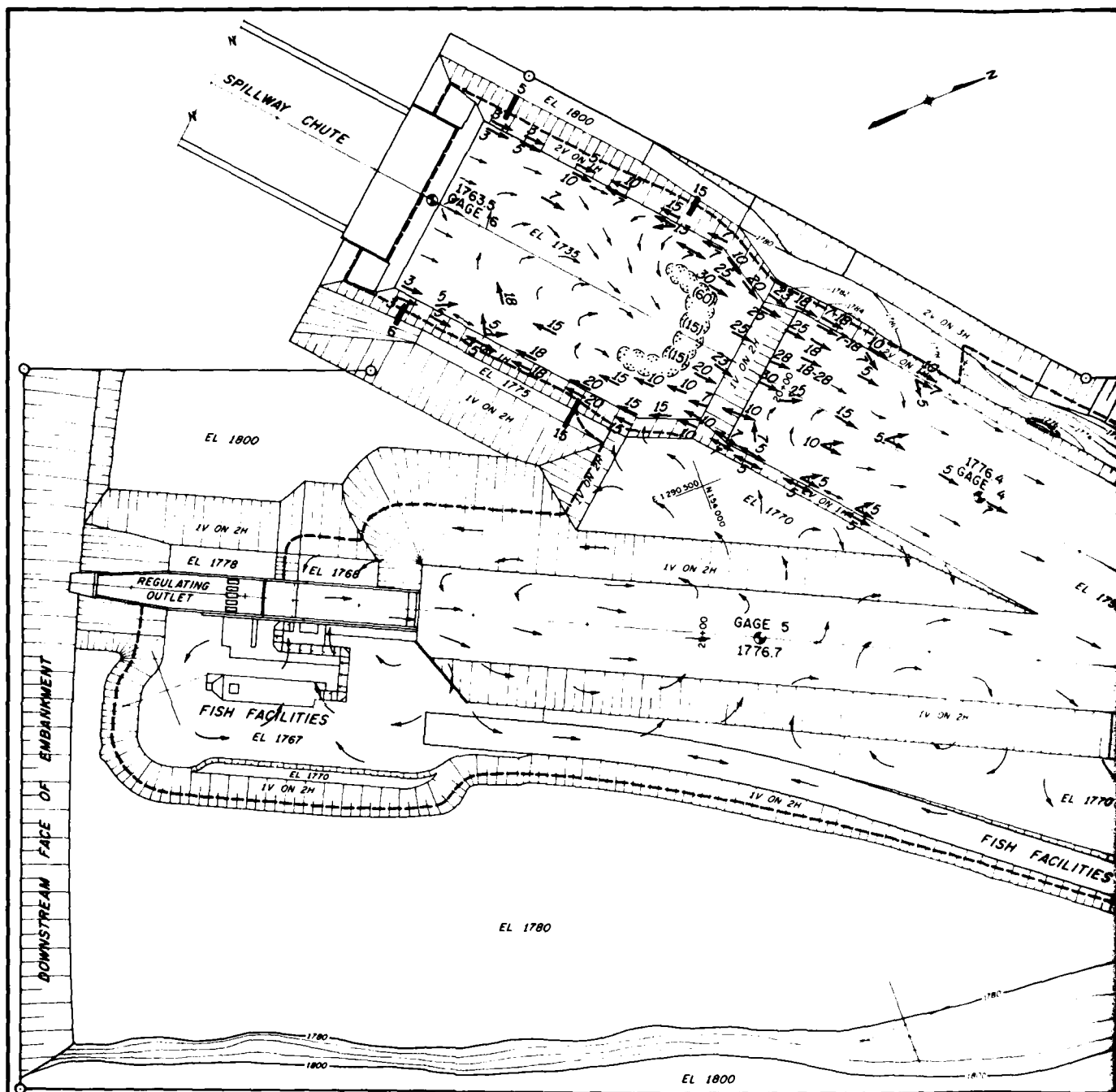
PLAN B PLUNGE POOL

FLOW CONDITIONS  
 SPILLWAY CHANNEL  
 RIVER DISCHARGE 49 800 CFS









RIVER DISCHARGE 35 600 CFS  
RIGHT BAY ONLY OPERATING



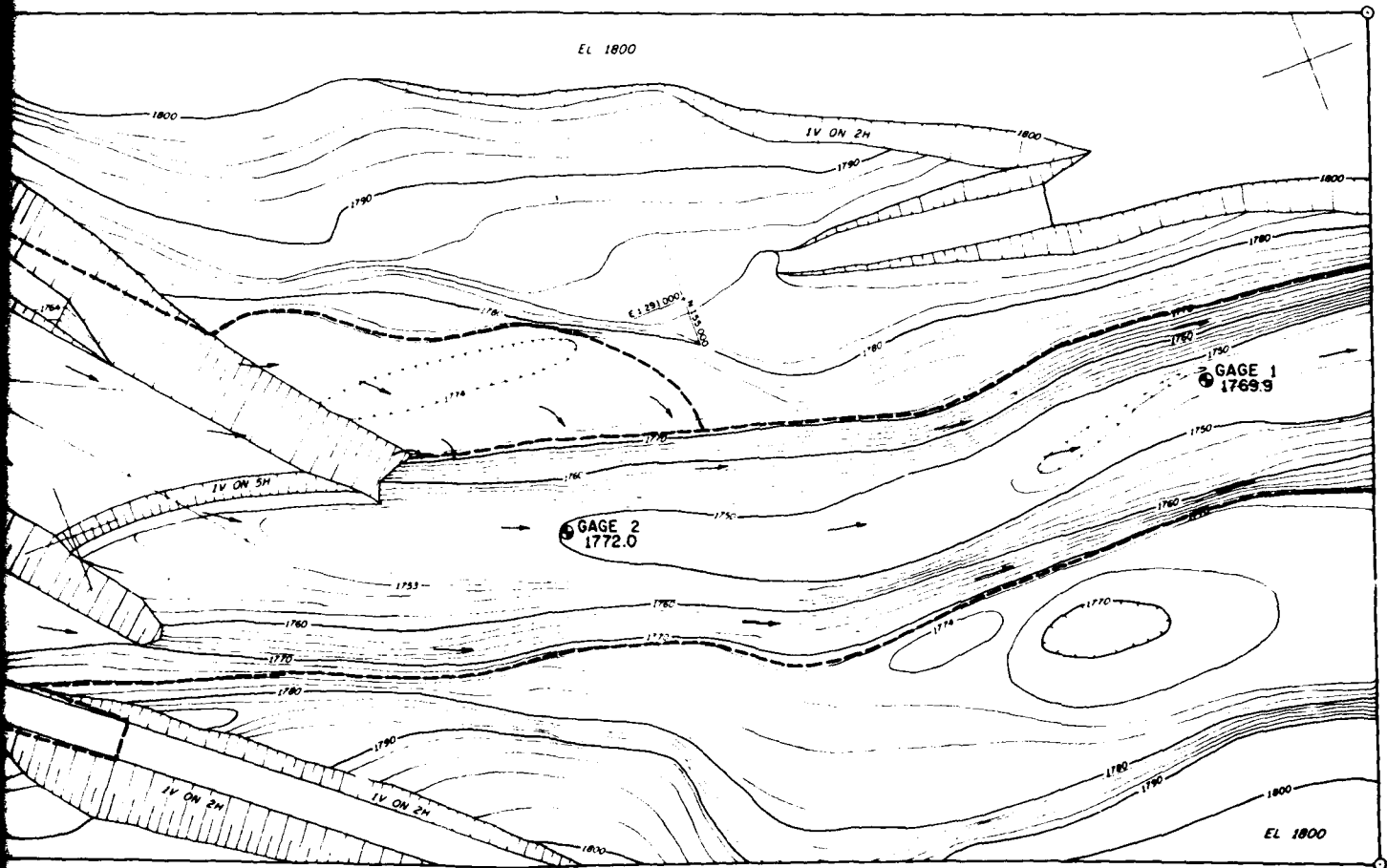
# **LEGEND**

-  VELOCITY IN FPS, 3 FT ABOVE BOTTOM
-  PRESSURE ON BOTTOM IN FT OF WATER
-  IMPACT AREA
-  WAVE RIDEUP IN FEET

# **OPERATING**

REGULATING OUTLET  
SPILLWAY

SCALE

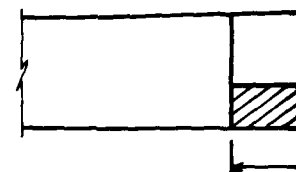
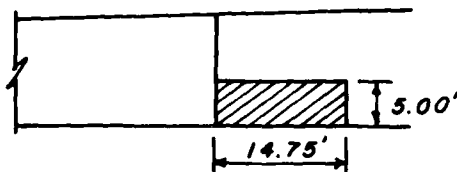
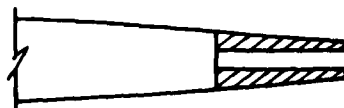


PLAN B PLUNGE POOL

FLOW CONDITIONS

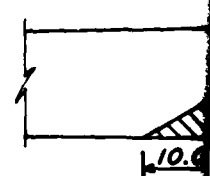
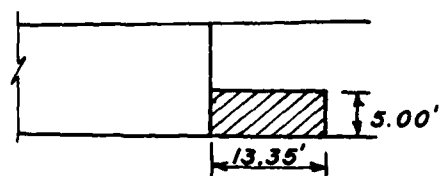
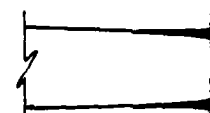
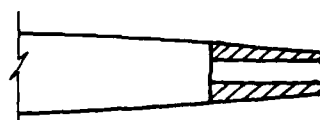
SPILLWAY CHANNEL

RIVER DISCHARGE 35600 CFS  
RIGHT BAY ONLY OPERATING



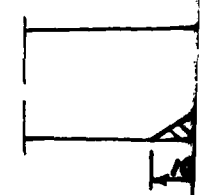
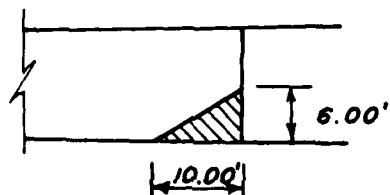
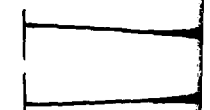
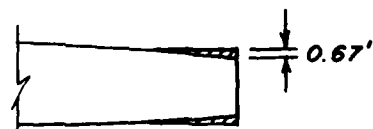
PLAN 1

PLAN 2



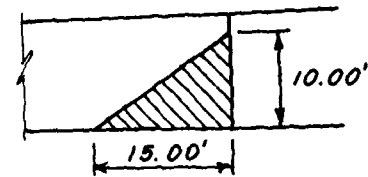
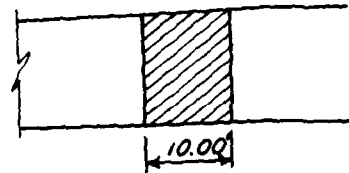
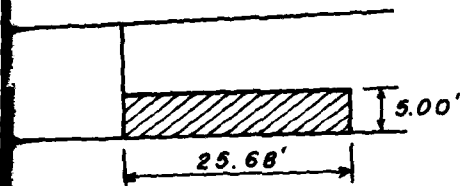
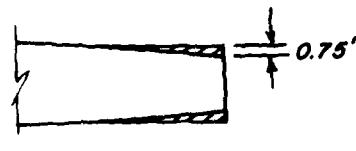
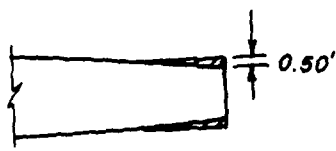
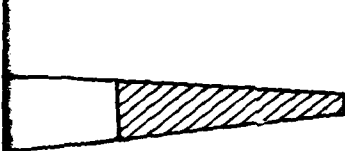
PLAN 5

PLAN



PLAN 9

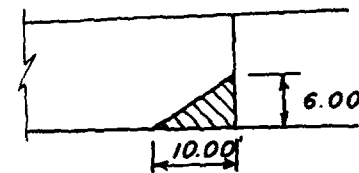
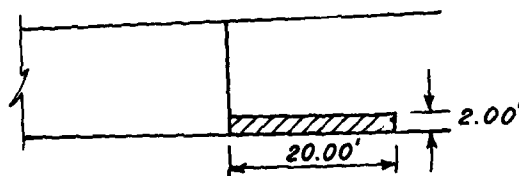
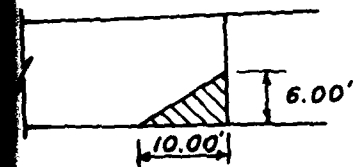
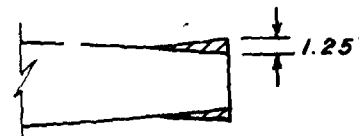
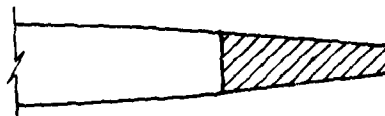
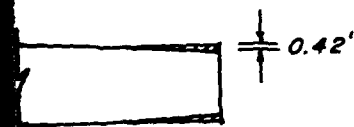
PLAN



PLAN 2

PLAN 3

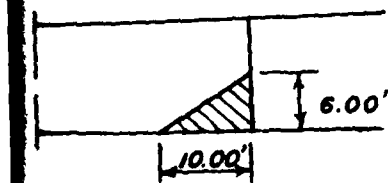
PLAN 4



PLAN 6

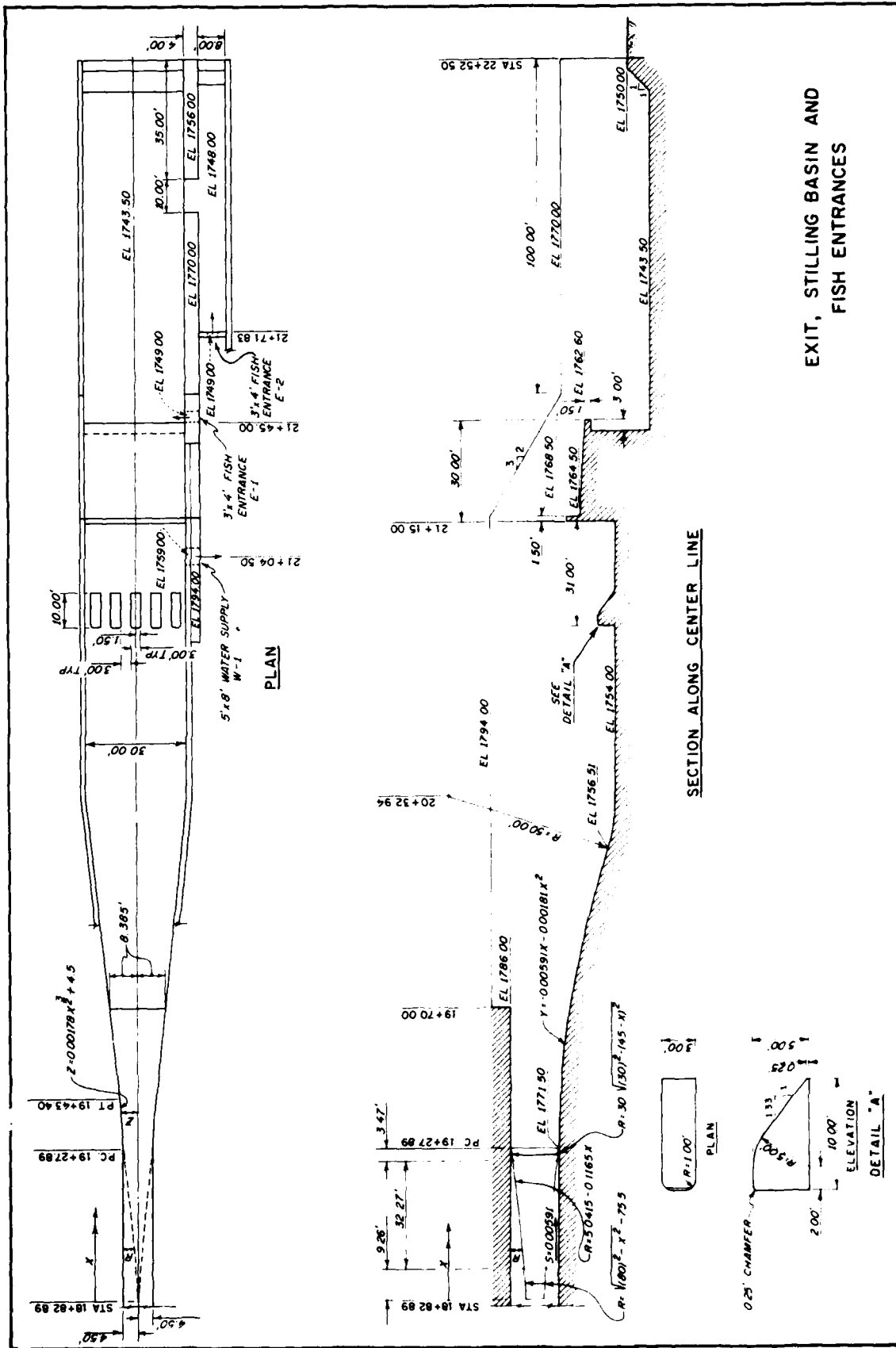
PLAN 7

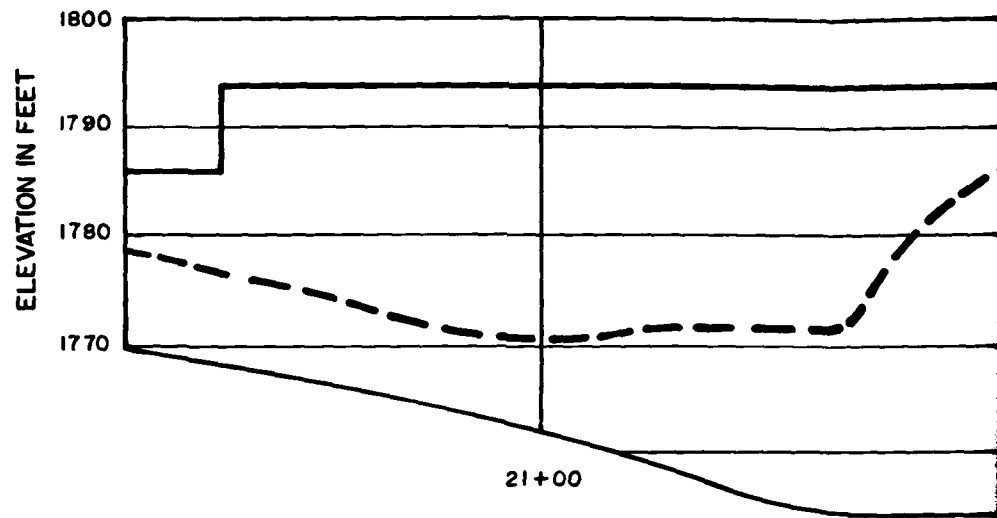
PLAN 8



PLAN 10

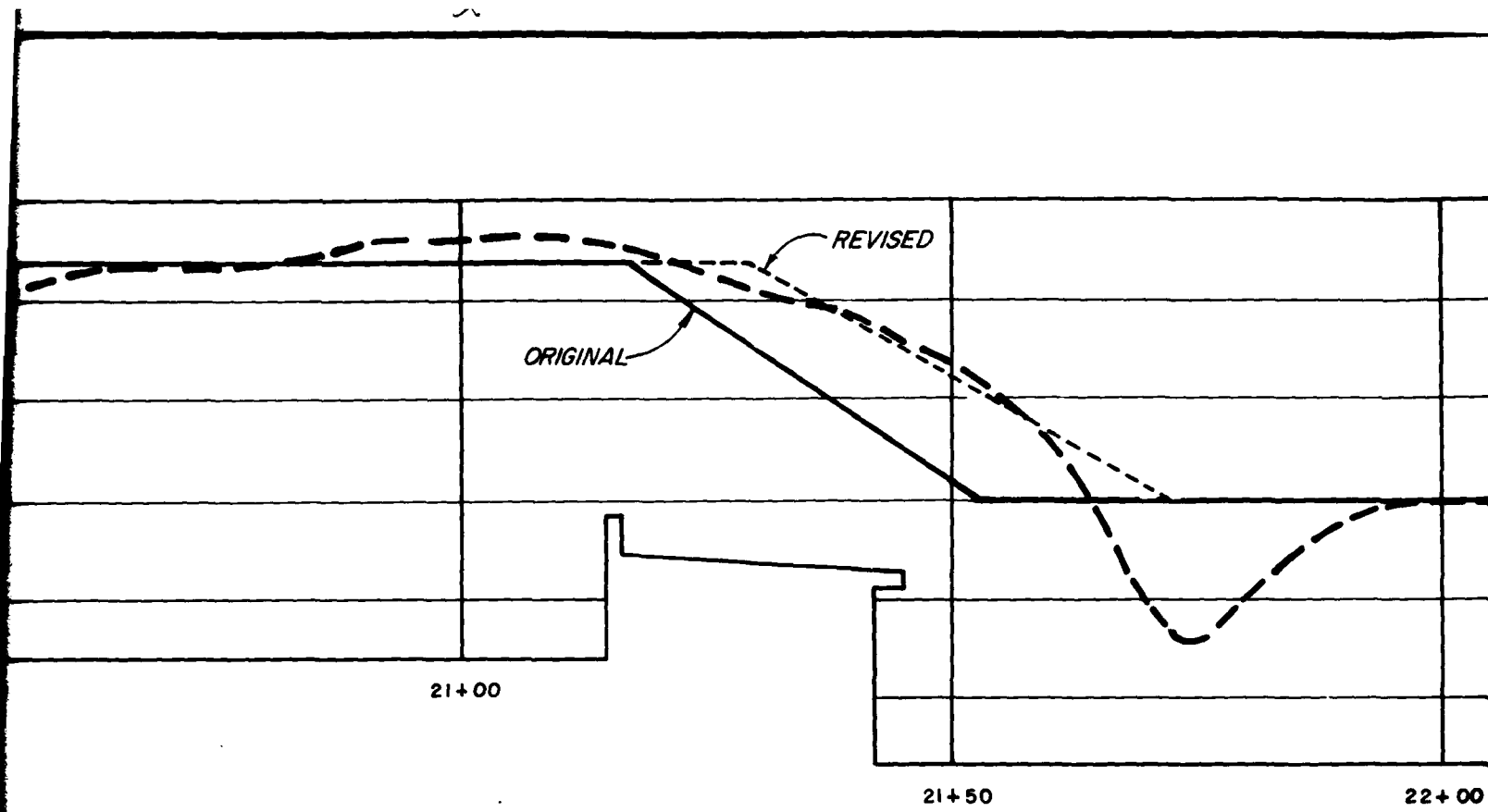
SPLITTER WALL  
PLAN DESIGNATION





NOTES

1. STILLING BASIN DETAILS SHOWN ON PLATE 3I.
2. TAILWATER ELEVATION 1761.83.

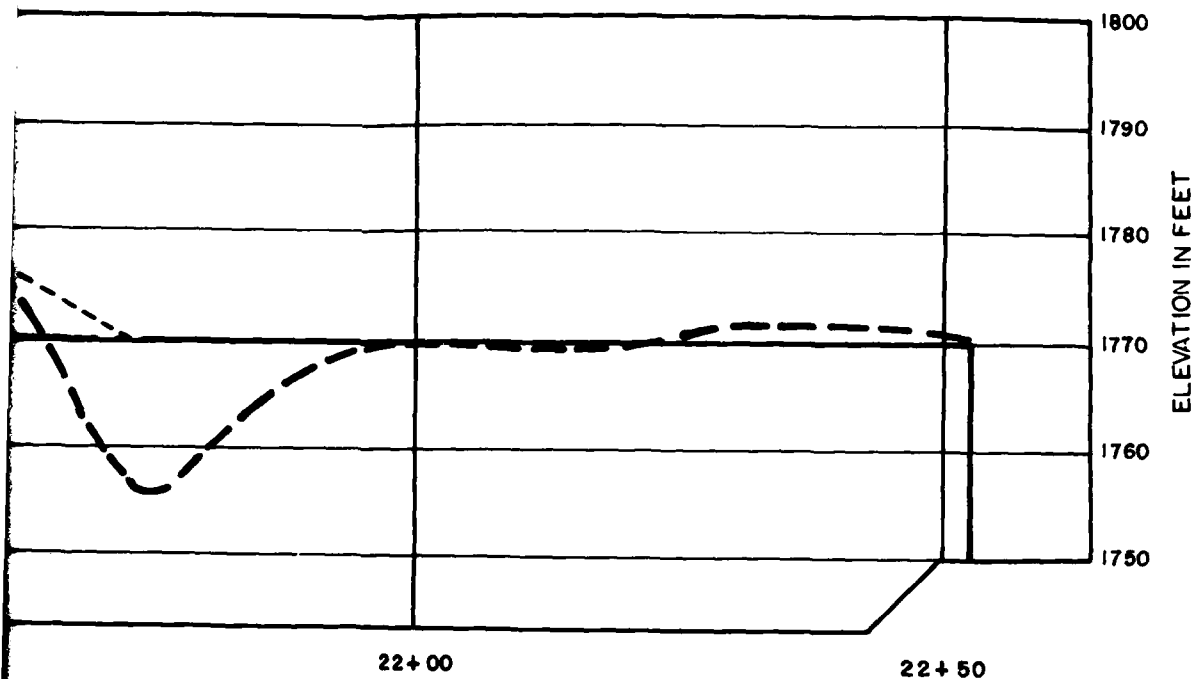


21+00

21+50

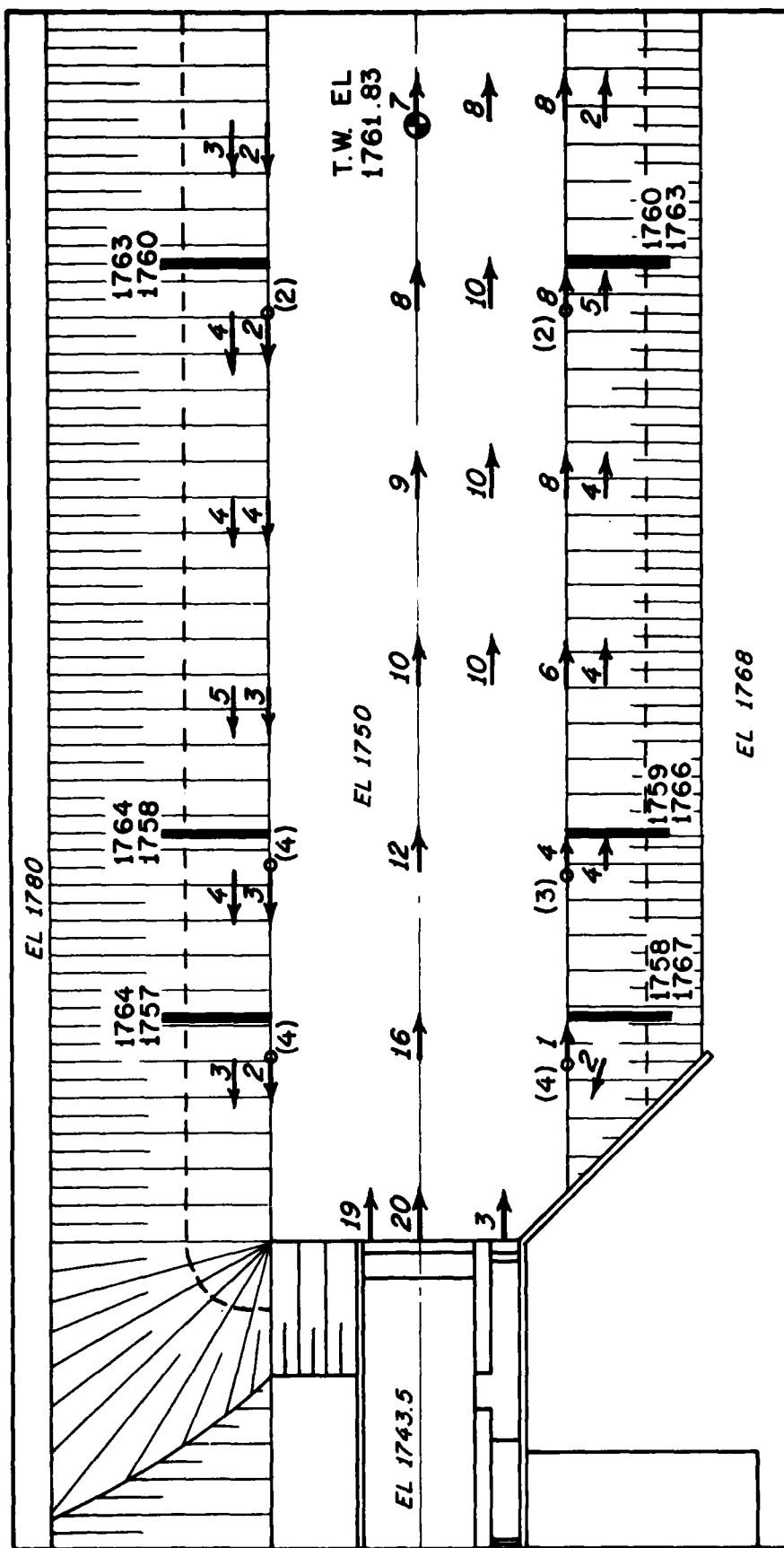
22+00

STATIONS



MAXIMUM WATER SURFACE PROFILE  
STILLING BASIN WALLS

DISCHARGE 6100 CFS  
CONDUIT ROUGHNESS  $K_s = 0.0002$  FEET



## LEGEND

**3**  
**←**  
**VELOCITIES IN FPS, 2 FT ABOVE BOTTOM**

**VELOCITIES IN FPS, 2 FT ABOVE BOTTOM**

**WAVE HEIGHTS IN FT**

**WAVE RIDEUP ELEV**

## REGULATING OUTLET CHANNEL

DISCHARGE 6100 CFS

**POOL ELEVATION 1987.0**

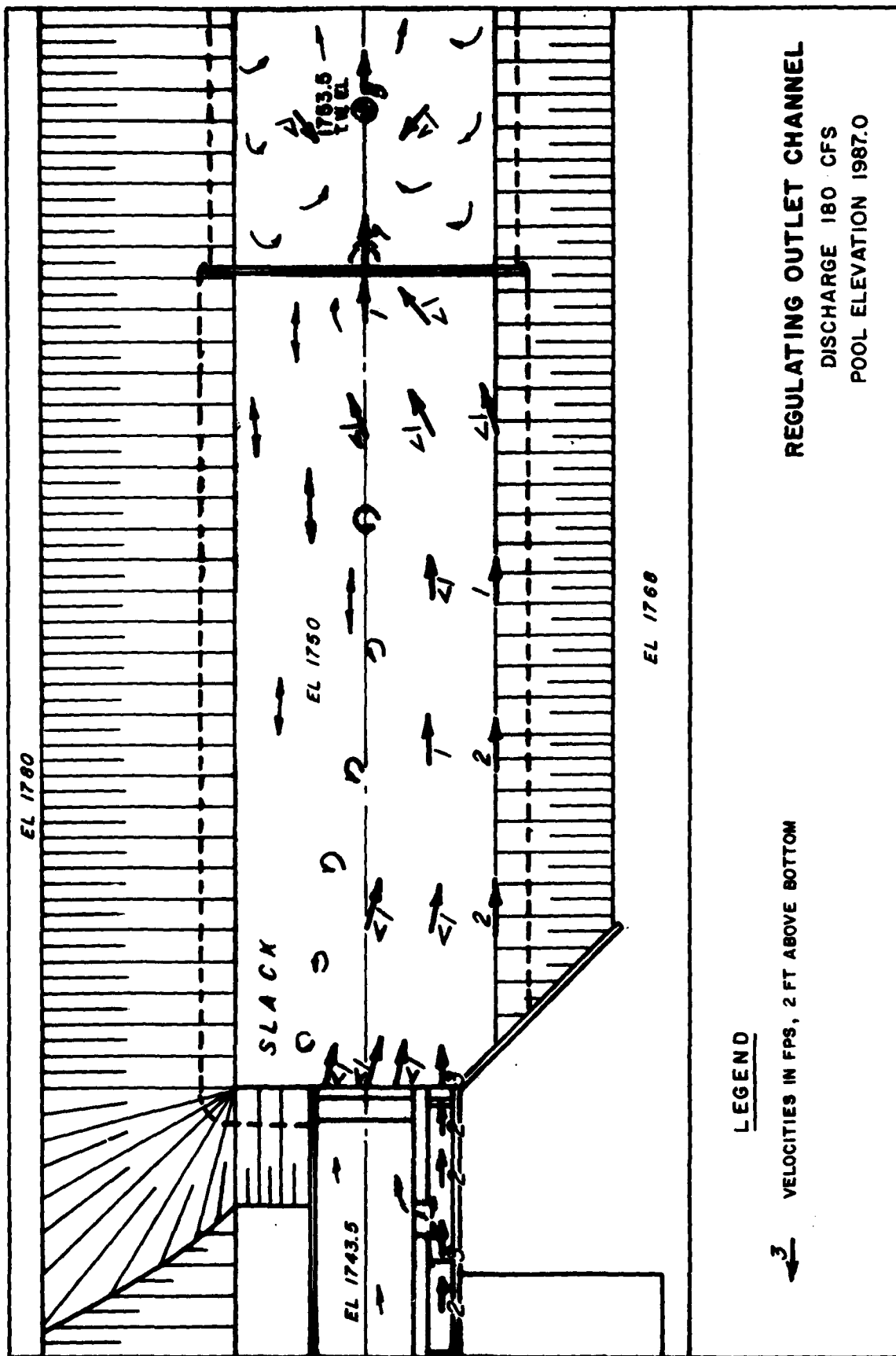


PLATE 34



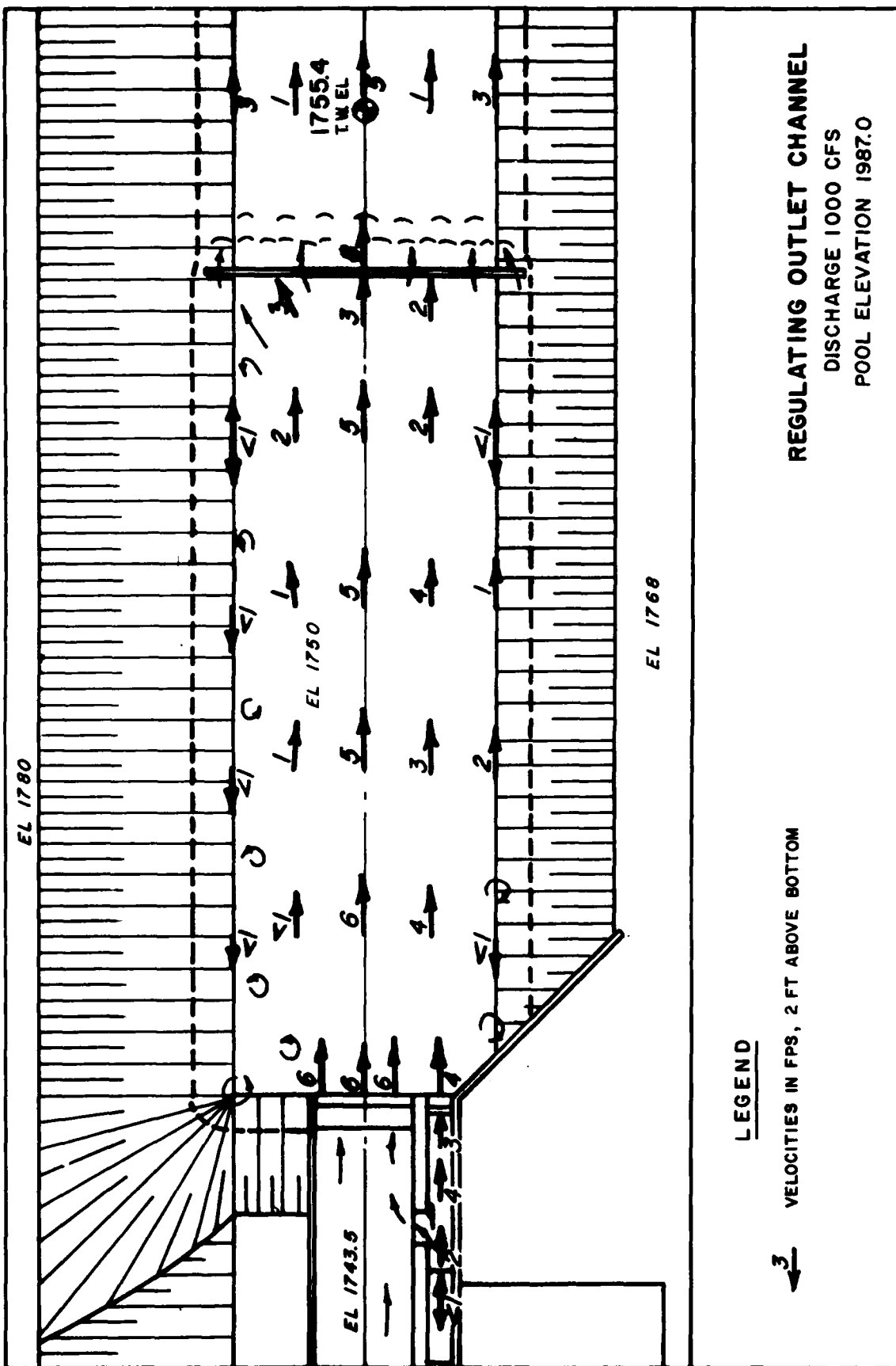
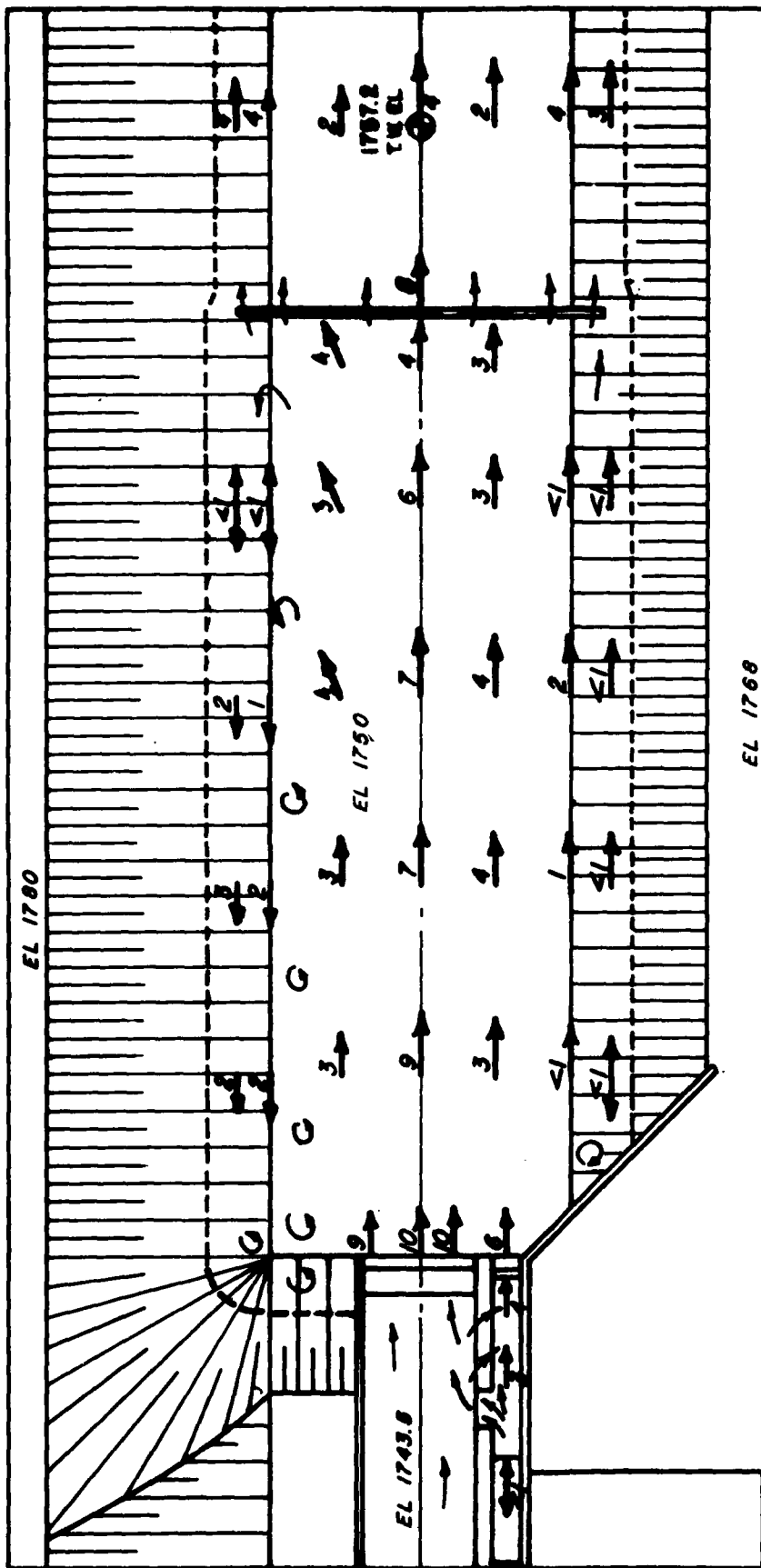


PLATE 36



# LEGEND

3 → VELOCITIES IN FPS, 2 FT ABOVE BOTTOM

## REGULATING OUTLET CHANNEL

DISCHARGE 2 000 CFS  
POOL ELEVATION 1987.0

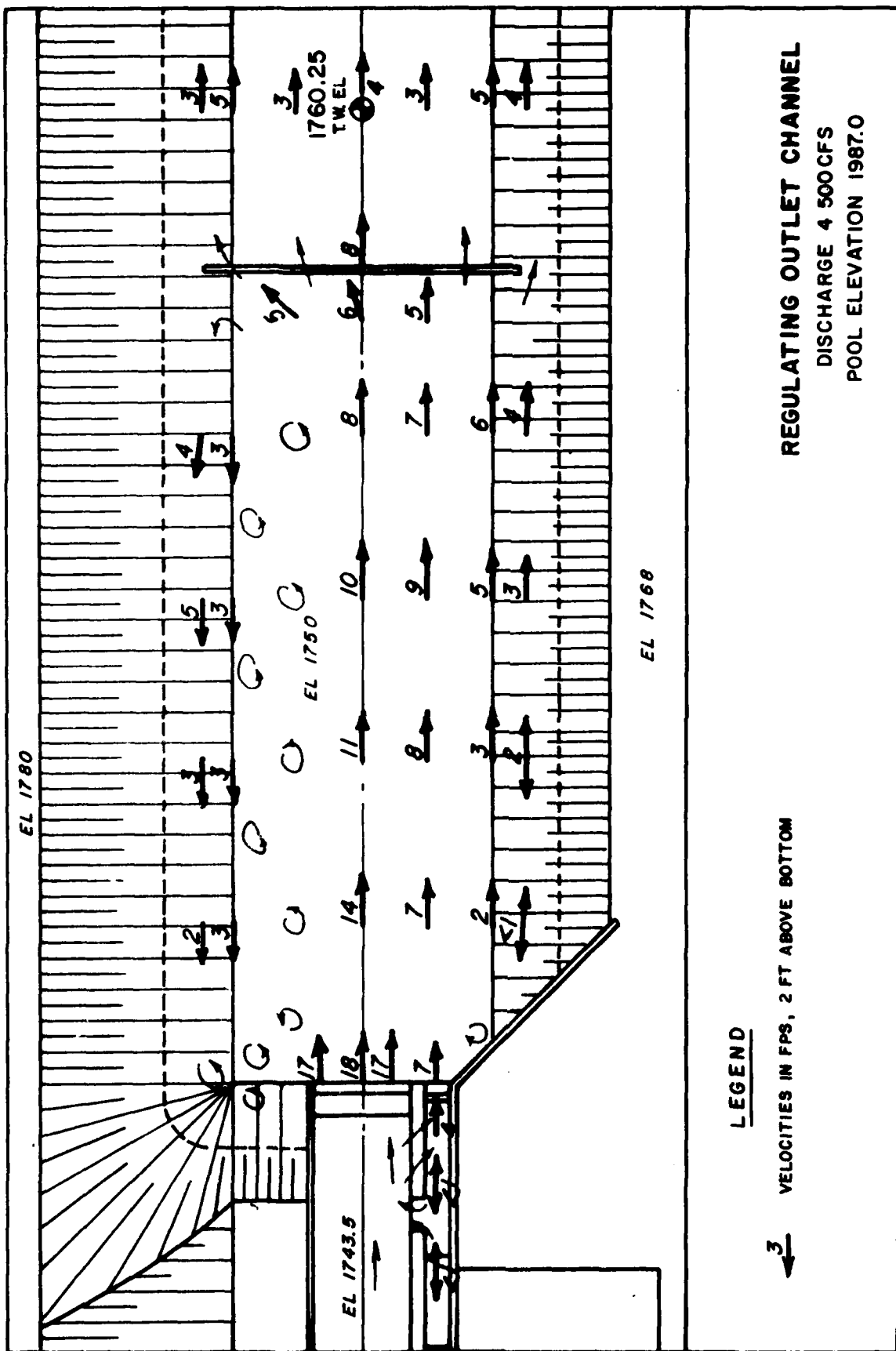
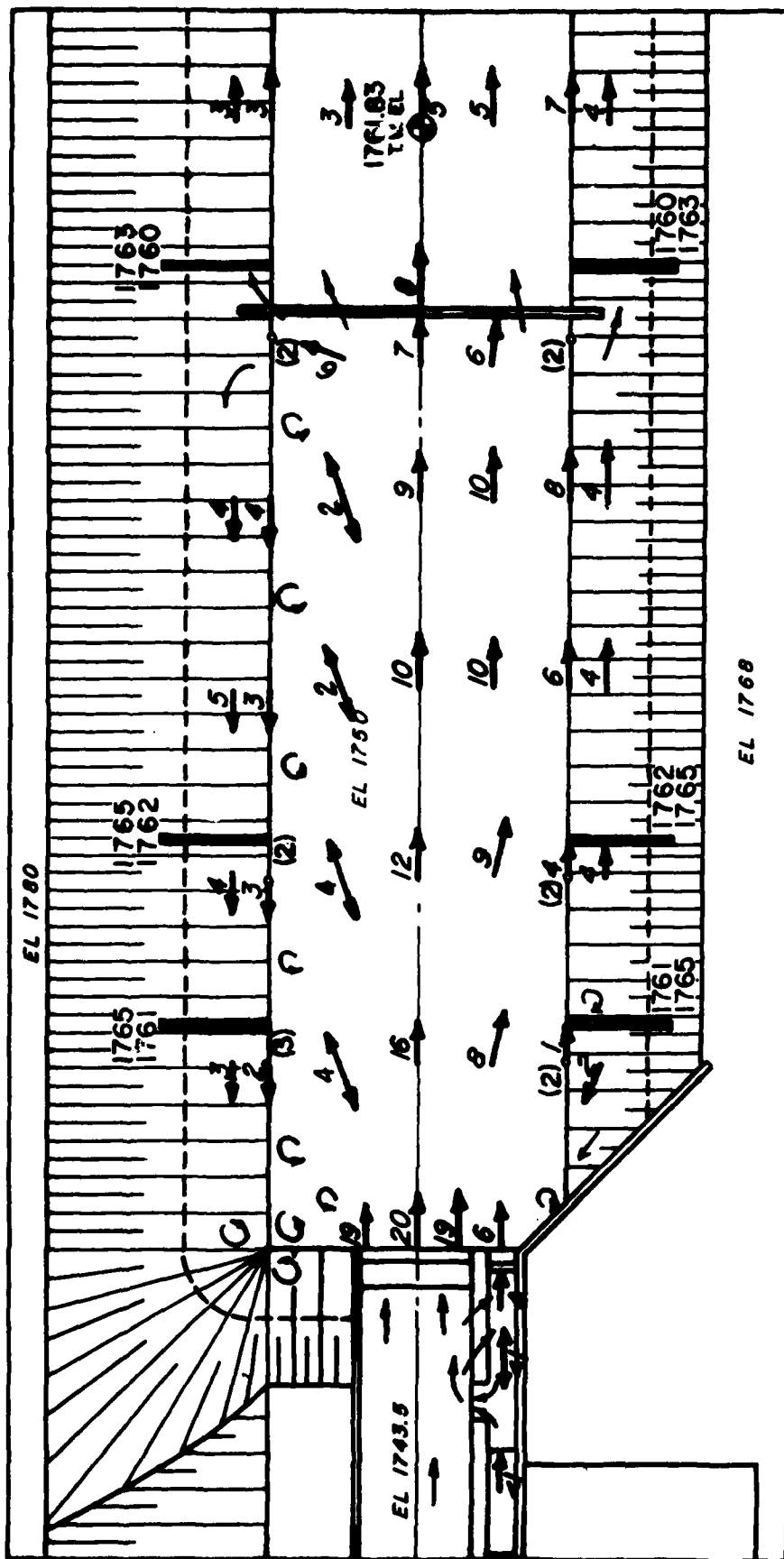


PLATE 38



# LEGEND

3  
 (4)°  
 1758  
 1767

VELOCITIES IN FPS, 2 FT ABOVE BOTTOM  
 WAVE HEIGHTS IN FT  
 WAVE RIDE UP ELEVATION

REGULATING OUTLET CHANNEL  
 DISCHARGE 6100 CFS  
 POOL ELEVATION 1987.0

EL 1780

EL 1743.5

EL 1750

EL 1753.5 T.W. EL

EL 1768

60-FT-WIDE CHANNEL

REGULATING OUTLET CHANNEL

DISCHARGE 180 CFS

POOL ELEVATION 1987.0

LEGEND

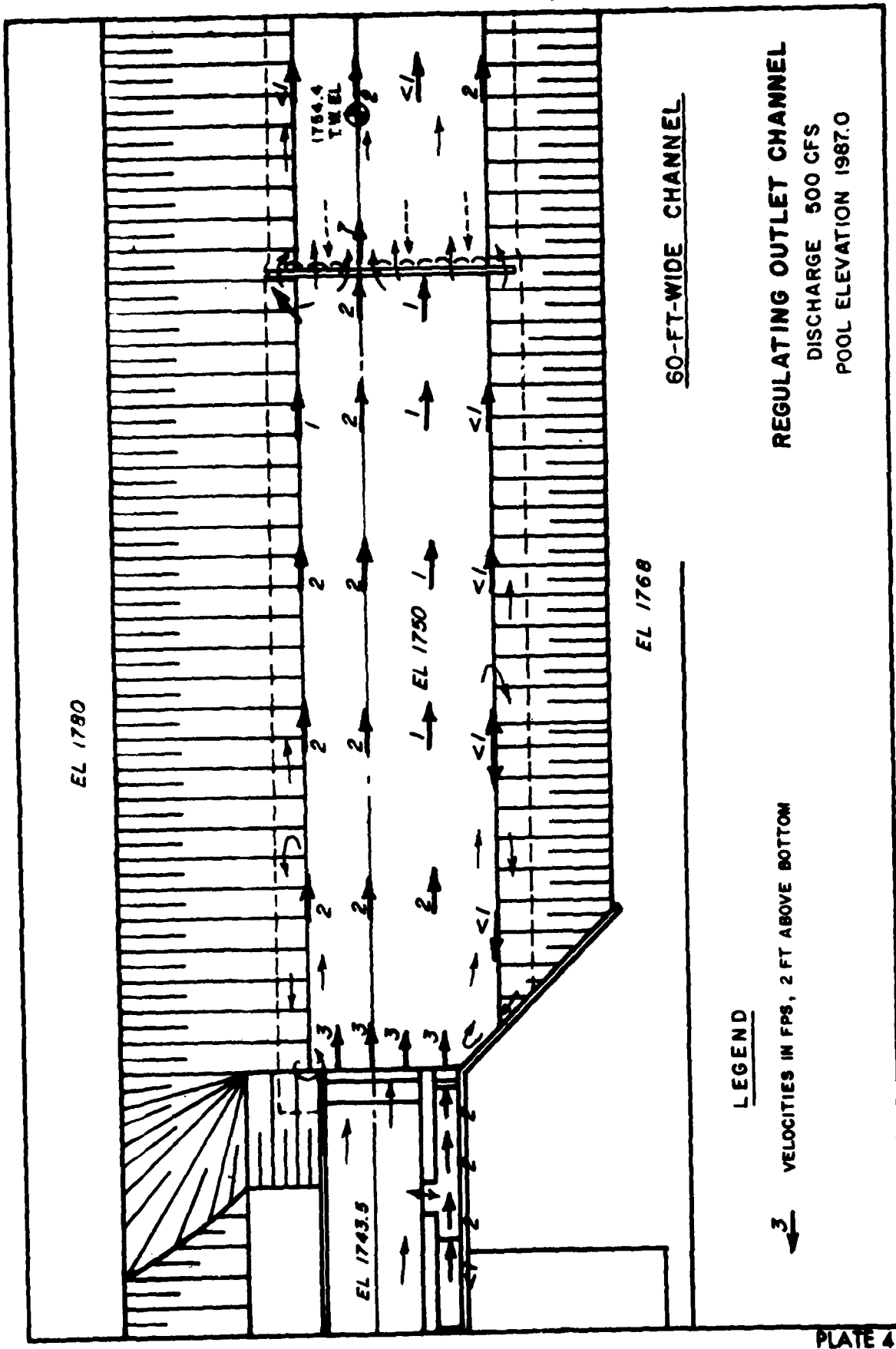
VELOCITIES IN FPS, 2 FT ABOVE BOTTOM

3

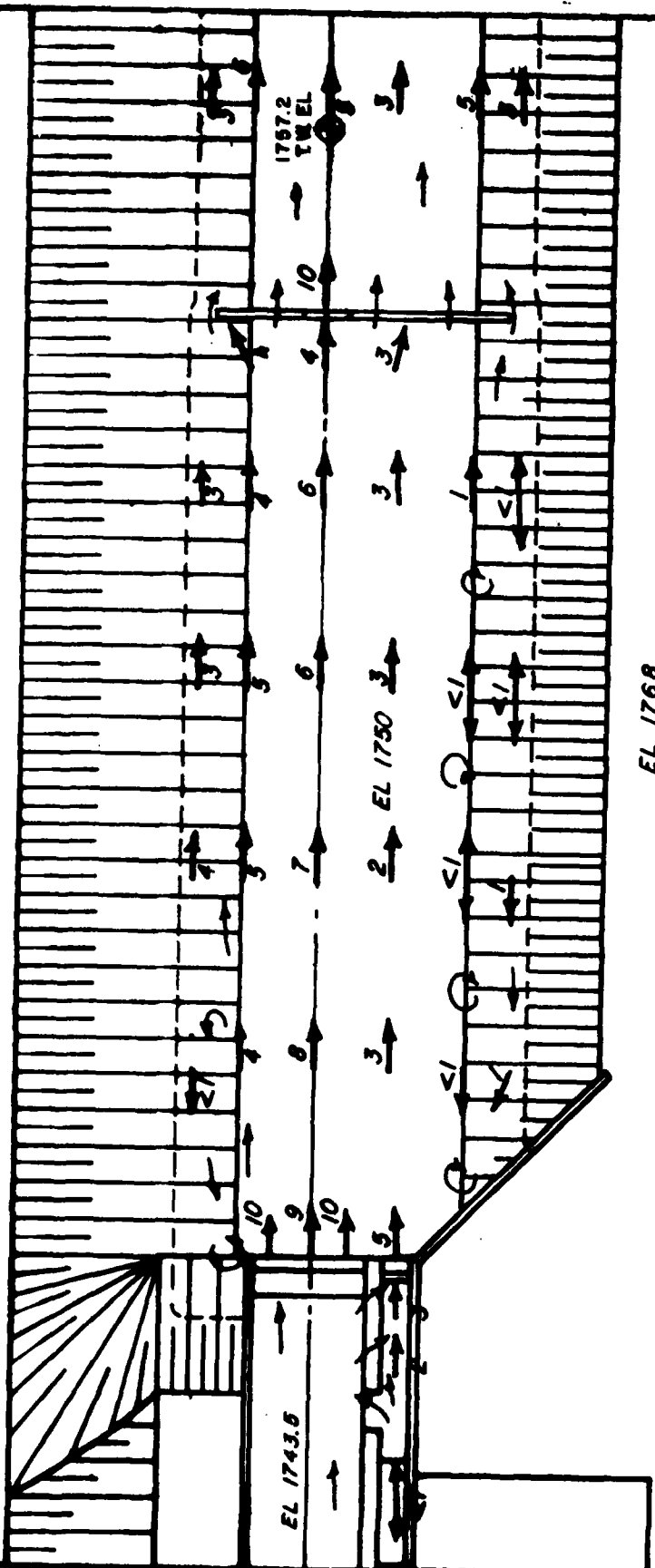
**3**  **VELOCITIES IN FPS, 2 FT ABOVE BOTTOM**

**REGULATING OUTLET CHANNEL**  
**DISCHARGE 180 CFS**  
**POOL ELEVATION 1987.0**

**60-FT-WIDE CHANNEL**



EL 1780



**EL 1768**

**60-FT-WIDE CHANNEL**

## REGULATING OUTLET CHANNEL

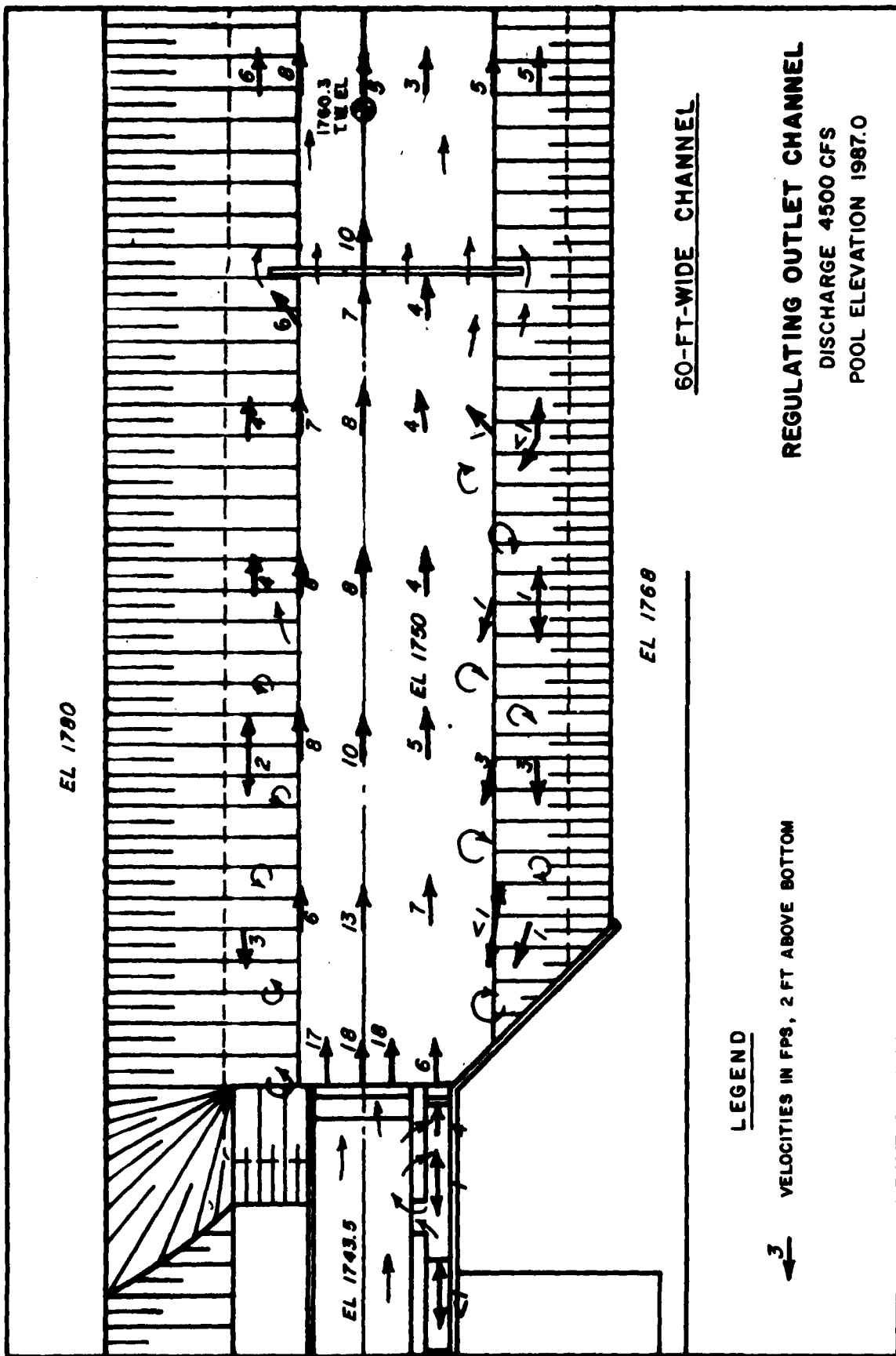
**DISCHARGE 2000 CFS**

**POOL ELEVATION 1987.0**

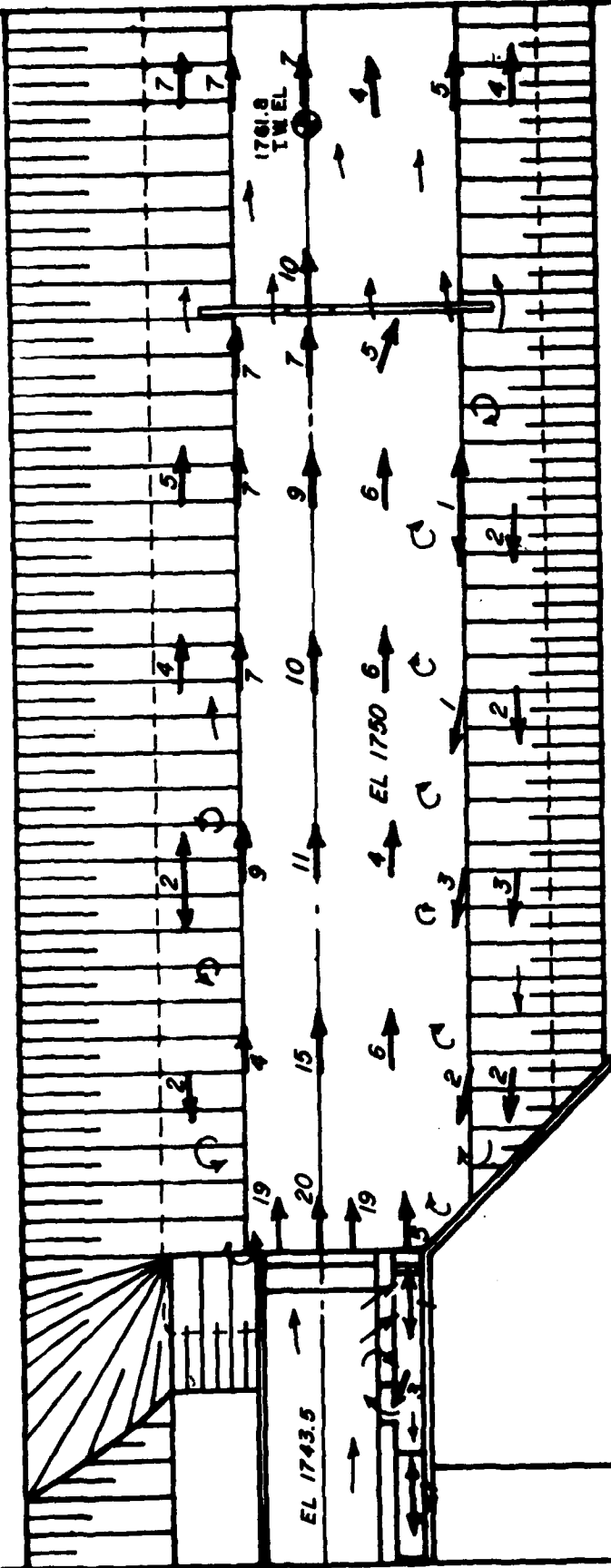
## LEGEND

**VELOCITIES IN FPS, 2 FT ABOVE BOTTOM**

3



EL 1780



EL 1768

LEGEND

3 VELOCITIES IN FPS, 2 FT ABOVE BOTTOM

REGULATING OUTLET CHANNEL

DISCHARGE 6100 CFS

POOL ELEVATION 1987.0

